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MODEL PILE BEHAVIOUR IN A CLAY SOIL

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled, MODEL PILE BEHAVIOUR IN A CLAY SOIL, submitted by Murray Carman Harris, in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The load-settlement behaviour of an axially loaded single pile in a purely cohesive soil has been studied by means of a model pile. Load transfer along the shaft and the base load have been measured by electrical resistance strain gauges.

An extensive review of available literature on bearing capacity, shaft resistance, pile instrumentation and model piles has been made. Some present day theoretical concepts of pile behaviour have been briefly outlined together with expressions required to analyse the behaviour of the model pile itself.

The model pile was made of one inch diameter brass tubing fifteen inches long with one inch and two inch diameter interchangeable bases. Strain gauges were placed in the vertical and circumferential directions at six levels on the inside of the pile. The base load was measured by a load cell. Four tests were made using a highly impermeable clay compacted around the pile in a cylindrical mould.

The tests showed that for the model pile the shaft load is fully developed at a much smaller penetration than the base load and at failure the shaft load can be considerably less than its maximum value. The distribution of skin friction was found to be uniform along the shaft at small loads but not necessarily so at greater loads. The average maximum skin friction was less than one third of the undrained shear strength of the shaft soil.

Recommendations for further study are given and emphasis placed on a need for detailed study of the nature of skin friction.

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GLOSSARY OF TERMS AND SYMBOLS

Adhesion: The shearing resistance between soil and another material under zero externally applied pressure. ASCE, 1958, p.4).

Base: The lowermost portion of a pile. Has diameter B and area A_B.

Base Resistance: The total load carried by the base of a pile, for any given pile load.

Bearing Capacity, (End Bearing Capacity): The maximum average load per unit area which the soil can support without rupture (Leonards, 1962, p. 539).

Bell: An enlargement of a pile at or near the base, formed by excavation or compaction methods. Usually cone (base down) or spherical in shape.

Cap, Pile cap: The top part of a pile at which the load is applied.

Cast-in-place pile: A concrete pile poured either with or without a casing in its permanent location, as distinguished from a precast pile (ASCE, 1946, p. 51).

Clay Soil, Clay: Fine grained soil or the fine grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents and which exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm. (.005 in some cases), but it is strongly recommended that this usage be discontinued; since there is ample evidence that from an engineering standpoint the properties described in the above definition are many times more important (ASCE, 1958, p. 8).

Coefficient of Variation, C_v: A measure of the relative standard deviation. Mathematically; $C_v = 100\ s/\bar{x}$ where s is the standard deviation and $\bar{x}$ is the arithmetic mean of the values.

Cohesion: The portion of the shear strength of a soil indicated by the term "c" in Coulomb's equation $\tau = c + \sigma \tan \varphi$ (ASCE, 1958, p. 10).

Cohesive Soil: A soil that when unconfined has considerable strength when air-dried, and that has significant cohesion when submerged. (ASCE, 1958, p. 10) A purely cohesive soil has a constant undrained shear strength regardless of the magnitude of the total confining stress.

Deviator Stress: The difference between the major and minor principal stresses in a triaxial test (ASCE, 1958, p. 14)

Effective Stress: The average normal force per unit area transmitted from grain to grain of a soil mass*. It is the stress that is effective in mobilizing internal friction. *(ASCE, 1958, p. 35).

Elastic: Capable of sustaining stress without permanent deflection; the term is also used to denote conformity to the law of stress-strain proportionality (Roark, 1954, pg 5).

End Bearing: (See Bearing Capacity)

General Shear Failure: Failure in which the ultimate strength of the soil is mobilized along the entire potential surface of sliding before the structure supported by the soil is impaired by excessive movement (ASCE, 1958, p. 31).

Hooke's Law: Linear proportionality of unit stress to unit strain

Load Capacity, Ultimate Load Capacity: The maximum total load on a pile required to cause continuous penetration

Local Shear Failure: Failure in which the ultimate shearing strength of the soil is mobilized only locally along the potential surface of sliding at the time the structure supported by the soil is impaired by excessive movement. (ASCE, 1958, p. 31)

Model Piles: Small scale piles used in research work to determine the behaviour and load capacity of piles.

Modulus of Elasticity, E: The rate of change of unit tensile or compressive stress with respect to unit tensile or compressive strain for the condition of uniaxial stress within the proportional limit. (Roark, 1954, p. 9) Various Moduluses of elasticity are (Morrison & Ford, 1955, p. 9):

a. Initial tangent modulus- the slope of the stress-strain curve at the origin

b. Secant modulus- the slope of a straight line from the origin to an arbitrarily chosen point on the stress-strain curve.

c. Tangent modulus- the slope of the stress-strain curve at an arbitrarily chosen point.

Neutral Stress, Pore Pressure: Stress transmitted through the pore water (water filling the voids of the soil)(ASCE 1958, p. 35).

Normal Stress: The stress component normal to a given plane, (ASCE 1958, p. 35)

Pile: Relatively slender structural element which is driven or otherwise introduced into the soil, usually for the purpose of providing vertical or lateral support (ASCE, 1958, pg 27).

Plane Strain: The two dimensional (biaxial) strain condition in which strain can occur in only two directions ($\epsilon_1 \neq 0$, $\epsilon_2 \neq 0$, $\epsilon_3 \neq 0$)(Jaeger, 1956, p. 60).

Plastic: A plastic material retains a permanent strain after removal of a load. A perfectly plastic material follows Hooke's Law up to the proportional limit and then begins to yield under constant stress. A perfect rigid plastic material does not strain until it reaches the yield point and then yields under constant stress.

Plastic Failure, Plastic Flow, Plastic Deformation: The deformation of a plastic material beyond the point of recovery, accompanied by continued deformation with no further increase in stress. (ASCE, 1958, p. 27).

Poisson's Ratio, μ: The ratio of lateral unit strain to longitudinal unit strain, under the condition of uniform and uniaxial longitudinal stress within the proportional limit (Roark, 1954, pg 10).

Principal Stress: Stress acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero. The largest, with respect to sign, is the <u>major</u> principal stress; the smallest is the <u>minor</u> principal stress, the stress neither the largest nor smallest is the <u>intermediate</u> principal stress.

Settlement, deflection: Vertical movement of pile cap, measured from initial position, under load.

Shaft: The embedded length of pile from ground surface to base or top of bell. Has diameter b, length H, and surface area A_s.

Shaft Capacity, Total Shaft Capacity, Q_s: The maximum load which can be taken by the pile shaft.

Shaft Resistance: The total load carried by pile shaft for any given pile load.

Shear Failure, Rupture: Failure in which movement caused by shearing stresses in a soil mass is of sufficient magnitude to destroy or seriously endanger a structure (ASCE, 1958, p. 31) See General Shear and Local Shear Failure.

Shear Stress: The stress component tangential to a given plane (ASCE, 1958, p. 36).

Skin Friction: The frictional resistance developed between soil and a structure (ASCE, 1958, p. 32).

Slickensides: Striations and polished surfaces along faulting or failure surface.

Standard Deviation: A measure of the average deviation from the mean. Mathematically

$$s = \sqrt{\frac{\Sigma(x^2)}{N-1}}$$ where x is the deviation from arithmetic mean and N is the number of samples.

Strain: The change in length per unit of length in a given direction (ASCE, 1958, p. 35).

Stress: The force per unit area acting within the soil mass. (See also effective stress, neutral stress, normal stress, principal stress, shear stress and total stress)(ASCE, 1958, p. 35).

Structure: The arrangement and state of aggregation of soil particles in a soil mass (ASCE, 1958, p. 33).

Total Bearing Capacity: The maximum total load on the base of a pile which the soil can support without rupture.

Total Stress: The total force per unit area acting within a soil mass. It is the sum of the neutral and effective stresses (ASCE, 1958 p. 36).

Undrained Shear Strength: The shearing resistance at failure for a soil from which no external drainage is permitted. For a purely cohesive soil numerically equal to one half of the deviator stress.

SYMBOLS

A_b	area of pile base
A_s	area of pile shaft
B	diameter of base
b	diameter of shaft
C	circumference of shaft
c	apparent cohesion intercept (for purely cohesive soil = τ_f)
c_a	adhesion
D	depth to bottom of base
E	modulus of elasticity
e	void ratio, base of natural logarithms
f	skin friction
f	(as subscript) failure condition
h	depth below soil surface
h	penetration of cone penetrometer
H	shaft length
I	influence value
N_c, N_γ, N_q	general bearing factors
p	total overburden pressure
p_o	effective overburden pressure
p_h	lateral pressure on side of shaft
Q_T	total pile load
Q_s	shaft load

Q_B	base load
Q_{max}	maximum pile load
q	unit base load
q_f	unit bearing capacity
q_a	allowable unit base load
s	standard deviation
t	thickness of pile shaft material
W	weight; pile, cone penetrometer
z	depth of rigid layer below base
α	shaft adhesion factor, f_{max}/τ_f
β	angle of equivalent free surface
ψ	cotangent of skin friction strain curve
γ	total unit weight of soil
γ'	adjusted unit weight of soil
γ_c	unit weight of concrete
Δ	penetration
ϵ	unit strain
θ	subscript for circumferential direction
μ	Poisson's ratio
ρ	soil heave
τ	shear stress
τ_f	shear stress at failure = shear strength
σ	unit normal stress
φ	angle of shearing resistance
$\emptyset$	surface movement

CHAPTER I

INTRODUCTION

1.1 General

The main purpose of a single load-bearing pile* is to distribute the stresses from a building load over a large volume of soil, or to transmit the load directly to a less compressible material at some depth below the surface. The design of piles attempts to provide an adequate factor of safety against shear failure* and to ensure that the pile will not settle excessively under the anticipated working load. For the purpose of design, piles are assumed to carry load either through shaft resistance* or end bearing* or a combination of both.

Conventional pile design practice is to assign an ultimate capacity to a pile on the basis of soil strength characteristics, load tests or pile driving formulae and to arrive at the working load by applying a suitable factor of safety. It is assumed that the pile deflections under the working load so obtained will be sufficiently low that no structural damage will result due to differential settlements* of the piles. In most cases the designer does not consider the settlement as such in his design.

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* Words or phrases marked with an asterisk (*) where they first appear are defined in alphabetical order in the Glossary of Terms and Symbols on pages ix to xii. A list of symbols is also given.

Except where the pile design has been proven by previous experience or where the design loads are small, it is preferable to prove the design by static load tests. Standard procedures have been established for such tests, for example ASTM Tentative Method D 1143-61T, "Test for Individual Piles Under Vertical Axial Load". Various building codes give the criteria by which the results are evaluated. The codes allow a maximum allowable net settlement or a maximum amount of settlement per ton of applied load at the working load, or require a factor of safety (usually 2.0) to be applied to the load which causes a disproportionate increase in settlement in the load test or some comparable criteria (Fletcher, 1962, pg 48)[1].

Design loads for driven piles are frequently evaluated, or construction is controlled, by the use of one of the many pile-driving formulae available. Although there is some justification for their use for cohesionless soils, pile-driving formulae are not recommended for use in cohesive soils (Chellis, 1961, pg 27) because of the temporary reduction of shaft resistance during the driving process due to excess pressure in the pore water of the soil.

The assumption that individual piles designed on the basis of an adequate factor of safety will result in a satisfactory foundation is not always justified. An example of this has been provided by the behaviour of a heavily loaded cement silo adjacent to a lightly loaded five story steel frame building.[2] The piles for both structures were

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1. References are listed alphabetically by author, first-named author when more than one or by issuing authority and date of publication in the List of References at the end of the text, page 130.

2. Dr. R.M. Hardy 1964, Private Communication

rammed, cast-in-place piles* carried in a deep uniform bed of clay. The piles had a satisfactory factor of safety against ultimate failure, but the difference in immediate or elastic settlements between the heavily and lightly loaded areas was sufficient to produce major structural damage at the junction of the two structures.

1.2 The Problem

The development of equipment capable of constructing large diameter cast-in-place piles in cohesive soils has caused increased research into rational methods of predicting not only the total load capacity* of a pile but also its settlement under the working load (Frischmann, 1962, pg 123). To be able to predict settlements from soil properties is particularly desirous for large diameter piles proposed to support heavy column loads, since full scale loading tests even up to the design load are very expensive. At present (1964) there is no rational design method substantiated by field test data to make such an analysis.

The problem is therefore, to attempt to evaluate those soil parameters which can be used to predict pile behaviour. Although the variability of soil properties of even the most uniform soil profile may prevent precise estimate of settlement, a rational design method could be used for preliminary design purposes, as a basis for extrapolating from field loading tests to piles carrying different design loads at the same site, and as a means for estimating differential settlements between piles carrying substantially different loads.

1.3 Purpose

The purpose of the thesis is to present a review of literature on the general subject of the behaviour of piles,

to develop such theory as is available to predict pile capacity and settlement, and to report on the results of model pile tests in a clay soil designed to demonstrate pile behaviour.

Except for the presentation of certain general pile theories the work is confined to the behaviour of individual axially loaded piles in a purely cohesive soil. Settlements considered are only those due to immediate compression of the soil and do not include consolidation or secondary compression effects, nor deep seated settlement, important as these may be.

1.4 Definitions

The definitions given in the Glossary of Terms and Symbols are in general those recommended by the American Society of Civil Engineers (ASCE, 1958); the symbols are those recommended by the International Society for Soil Mechanics and Foundation Engineering. Specific symbols are defined where they first appear in the text.

Attention is drawn to the definitions for pile load capacity and end bearing or bearing capacity. The word "bearing" is used herein to refer only to the resistance developed by the base* of a pile or footing and not in the more general sense, as found in many publications and texts, of the overall load capacity of a pile.

The word "total" is used to indicate a force; where it is omitted, as in "end bearing capacity", a unit force, eg. $kg/cm.^2$, is indicated.

The letter f is used to indicate the skin friction* on a surface, whether adhesion* or true friction, except when used as a subscript, in which case it refers to a failure condition, eg. τ_f means the shearing stress* at failure.

The term "cohesive soil" is one subject to many interpretations; herein it is used in the general sense to indicate any soil possessing some cohesion* or shearing resistance when submerged in water. For a soil exhibiting constant undrained shearing resistance regardless of the magnitude of the principal stresses* (the particular subject of study herein) the term "purely cohesive" is used; the term "clay soil*" will be considered synonymous with "purely cohesive soil".

The word "elastic" used herein is intended to describe a material in which shearing stress increases with strain* as opposed to plastic* behaviour in which strain may increase without a corresponding increase in stress and in which the stress is a function of the rate of strain. Cohesive soils cannot be considered ideal elastic solids because they show substantial change in properties with change in stress history and exhibit residual strains upon the removal of a stress. However for the purpose of estimating pile behaviour at working stresses (approximately one-third of plastic failure*) the soil will be assumed to act not only as an elastic solid but also to have a constant modulus of elasticity and Poisson's ratio* (Terzaghi, 1943, pg 367). The justification for this lies only in the success by which theory based on such an assumption is able to predict pile behaviour.

CHAPTER II

LITERATURE REVIEW

2.1 Scope

The purpose of this chapter is to survey the available background literature on the behaviour of piles. The topics reviewed are end bearing capacity, shaft resistance, field pile test methods and model pile* testing. Although certain theories and methods are ascribed to definite authors it should be appreciated that they have all relied more or less on workers here unnamed. All formulae given are in the symbols used herein and not necessarily those used by their originator.

2.2 Bearing Capacity

The earliest methods for calculating end bearing capacity of piles were those proposed for footings at shallow depths or at the surface. Among the first was the expression developed by Bell (1914) which considered active and passive zones below a long footing. The analysis assumed that an active zone of pressure was formed below the base and that this was resisted by a passive zone beyond the footing edge. The soil was considered to have both cohesion and an angle of shearing resistance. The analysis did not consider the weight of the soil and thus the result was independent of the footing size.

In 1920 Prandtl developed a more rigorous analysis of bearing capacity when analysing the indentation hardness

of metals (Leonards, 1962, pg 540). The failure mechanism was that for a purely cohesive, perfect rigid plastic, weightless and semi-infinite material acted upon by a smooth, rigid, strip footing (die). A basic assumption of Prandtl's solution was that the footing was guided (as a metal punch would be) and that therefore the footing would move vertically downward (Tschebotarioff, 1951, pg 221). The penetration of the footing was assumed to form a wedge of material below the base bounded by symmetrical zones of radial shear and plane shear, the latter zones intersecting the plane of the base, as shown in Figure 2-1. At failure the unit load for a footing on the surface, ie. the bearing capacity q_f, was found as

$$q_f = (2+\pi)\ c \qquad \text{2-1}$$
$$= 5.14\ c$$

where c is the undrained shear strength* of the soil(for Prandtl the shear strength of the metal).

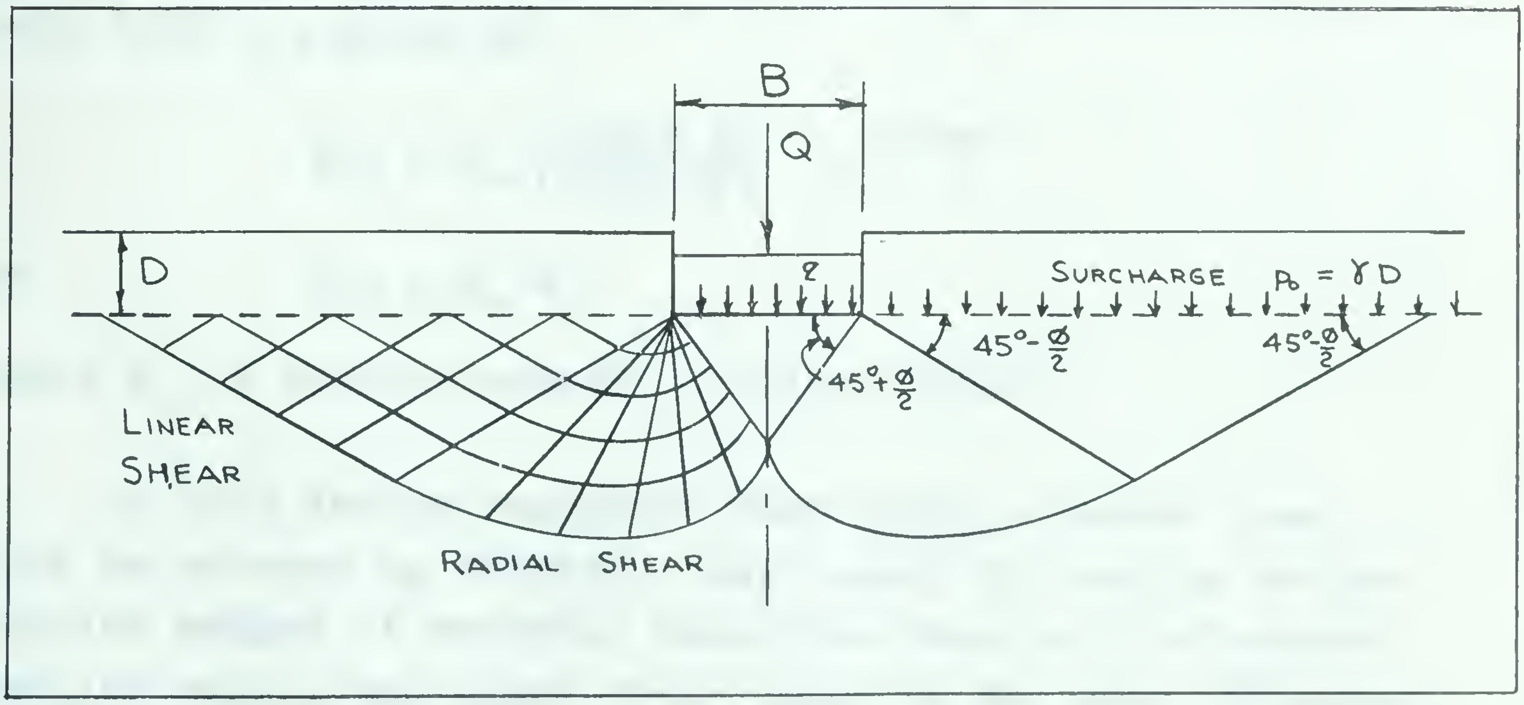


Figure 2-1. Prandtl zones of plastic equilibrium for loaded strip footing of width B.

In 1921 Prandtl modified his previous analysis to account for materials which fail in accordance with Coulomb's law (possessing both cohesion c and an angle of shearing resistance φ):

$$\tau_f = c + \sigma \tan \varphi \qquad 2\text{-}2$$

where τ_f is the shear resistance at failure and σ is the normal stress* on the failure plane (Tschebotarioff, 1951, pg 221). The resulting equation was (R.R.L., 1952, pg 453):

$$q_f = \frac{c}{\tan \varphi} \left[\frac{1+\sin \varphi}{1-\sin \varphi} \cdot e^{\pi \tan\varphi} - 1 \right] \qquad 2\text{-}2a$$

or $$q_f = c\, N_c \qquad 2\text{-}2b$$

where the symbols are as given previously, N_c is a general bearing factor, and e is the base of natural logarithms.

The effect of the surcharge p_o shown in Figure 2-1 was accounted for by Reissner in 1924 as an additional unit load q'_f given by:

$$q'_f = p_o \frac{(1+\sin \varphi)}{(1-\sin \varphi)} \cdot e^{\pi \tan\varphi} \qquad 2\text{-}3a$$

or $$q'_f = p_o\, N_q \qquad 2\text{-}3b$$

where N_q is another general bearing factor.

In 1923 Hencky suggested that under a smooth base, such as assumed by Prandtl, there would in reality be two smaller wedges of material below the base with attendant smaller radial and plane shear zones at the side (Meyerhof, 1955, pg 229). However the bearing capacity has the same value as given by equation 2-1. Hencky also analysed

Prandtl's bearing capacity problem for a circular base on a purely cohesive material and found that (Marwick, 1942, pg 84):

$$q_f = 5.64\ c \qquad 2\text{-}4$$

where the symbols have the same meaning as in equation 2-1.

The various analyses of Prandtl, Hencky and Reissner were based on failure patterns or slip line fields as deduced by plastic failure analysis, assuming negligible strains at the point of plastic failure and a rigid, weightless material. A different approach, that of assuming a failure surface in accordance with field observation and analysing the forces along this boundary, appeared in the late 1920's.

Hogentogler and Terzaghi (1929, pg 51-52) gave a method of calculating bearing capacity based on the assumption of a wedge of material forced down and sideways below the base which in turn forces a wedge of soil upward

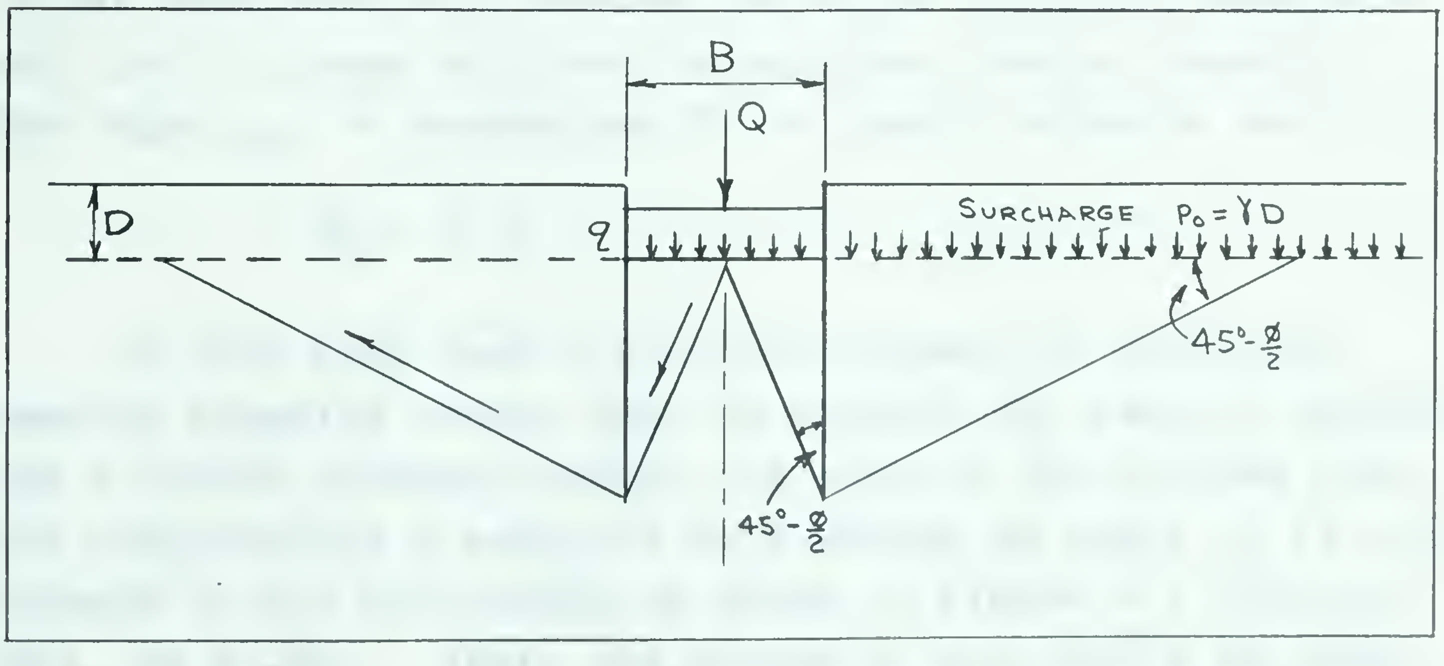


Figure 2-2 Hogentogler-Terzaghi Bearing Capacity Diagram

and outward under a surcharge p_o, as shown in Figure 2-2 (this analysis is quite similar to that of Bell).

From the statics of the base and wedge forces, assuming the soil to be in a state of plastic equilibrium, the following expression was derived equating the bearing capacity q_f in terms of the cohesion c, angle of shearing resistance φ, unit weight γ, width of base B and surcharge p_o:

$$q_f = p_o \cot^4\alpha + B\gamma \cot\alpha \, (\cot^4\alpha - 1) + 2c \frac{\cos\alpha}{\sin^3\alpha} \qquad 2\text{-}5$$

where $\alpha = 45^o - \varphi/2$

The analysis is for a long footing and assumes no shearing stresses on the base, ie. a perfectly smooth base.

Fellenius in 1929 took as the failure surface a circular arc passing through one edge of the footing (Golder, 1942, pg 451)(Skempton, 1942, pg 316). This failure surface assumes that the footing fails by tilting about a centre determined by trial and error procedure, as in the Swedish-circle method for slope stability analysis; the circle chosen to give the smallest bearing capacity. The value q_f, so determined for a purely cohesive soil is:

$$q_f = 5.52 \; c \qquad 2\text{-}6$$

In 1932 Krey used a similar approach to calculate bearing capacity except that he assumed the failure surface was a circle passing through one edge of the bearing area and intersecting a wedge of soil making an angle of $45 - \varphi/2$ degrees to the horizontal as shown in Figure 2-3.(Wilson, 1941, pg 87-88). Again the centre of the circle is found by trial and error but with the centre of the circular arc

located along the plane of the base. For a surface load the bearing capacity on a purely cohesive soil would be (Wilson, 1941, pg 94):

$$q_f = 6.05\ c \qquad 2\text{-}7$$

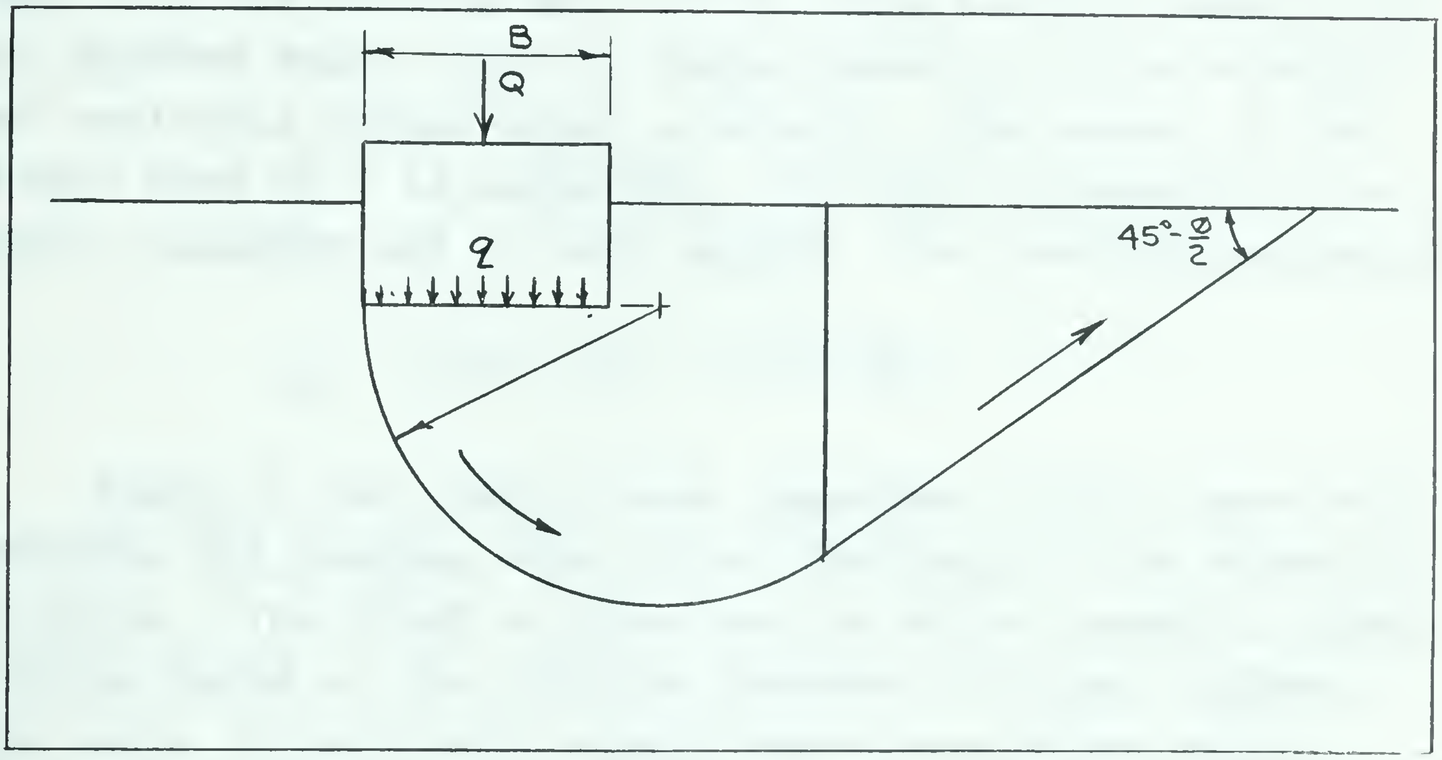


Figure 2-3 Krey Bearing Capacity Diagram

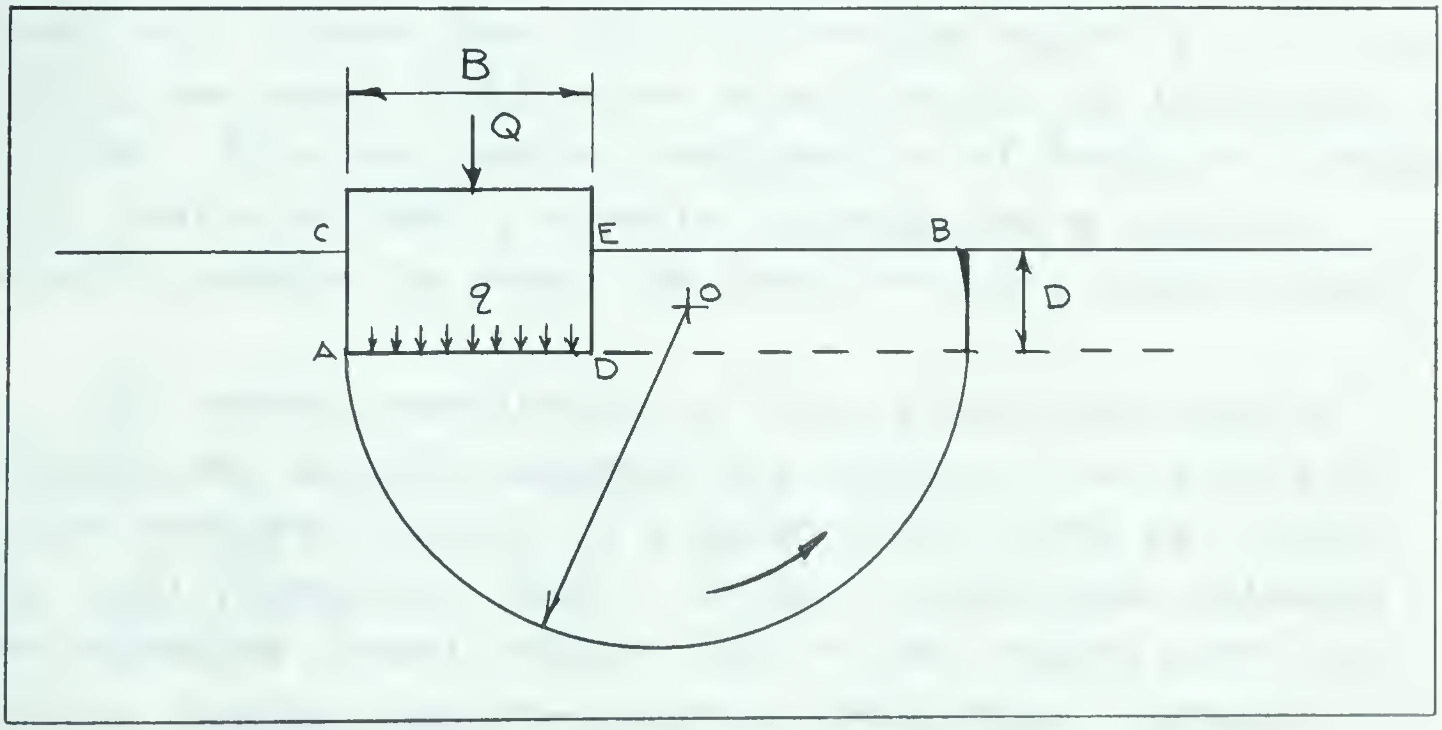


Figure 2-4 Modified Fellenius Method (1941)

Wilson (1941, pg 89-91) reports on a modification of the Fellenius method for footings at depths up to about 1 1/2 times the width. The failure surface is assumed to be a circle passing through one edge of the base and continuing vertically upwards to intersect the soil surface, as shown in Figure 2-4. The soil is assumed to have shrunk away from the footing wall at AC. The bearing capacity is the minimum value found by taking moments of the rotating and resisting forces about a point O. The moment of the shaded area at B is neglected. The soil is assumed to be purely cohesive and to have weight. The resulting equation is

$$q_f = 5.52\ c\ (1 + 0.377\ \frac{h}{B}) \qquad 2\text{-}8$$

Early in the 1940's three important studies were made relating the bearing capacity of footings to the behaviour of piles. The first of these was by Golder based on model footing tests at the Building Research Station, England. The tests of particular significance were those using rectangular steel footings of various length to width ratios on saturated clay soils. The result of tests to failure on these soils showed that the unit bearing capacity of a square footing was about 30% greater than that for an infinitely long footing. This was partial confirmation of Hencky's findings which indicated that a circular footing had a bearing capacity greater by about 10% than that of a long footing.

The second contribution of this period was that of Skempton who in 1942 analysed the failure of an 8 foot by 9 foot concrete footing at a depth of 5.5 feet in a soft clay soil (Skempton, 1942). In his calculations relating the estimated actual failure load to the values given by various bearing capacity theories (Fellenius, Terzaghi,

Hencky), Skempton corrected the bearing capacity formula for both the surcharge due to the depth of the footing, γD, and the skin friction developed along the footing wall with the resulting equation:

$$q_f = 5.64\ c + \gamma D + \frac{F}{A} \cdot c' \qquad 2\text{-}9$$

where c = cohesion of soil below base
c' = cohesion of soil along shaft
γ = unit weight of soil above base
D = depth of footing
F = area of footing wall against soil
A = area of base

By far the most important contribution in these years was by Terzaghi, who, in his textbook, "Theoretical Soil Mechanics" (1943) provided the first rational analysis of pile or pier load-carrying capacity. Terzaghi analysed the behaviour of a deep foundation in terms of Prandtl's analysis for a surface footing plus the effect of the shearing forces along the pile surface and on an outer cylindrical shear boundary EF shown in Figure 2-5. The resulting equation for a circular base gives a bearing capacity of (Terzaghi, 1943, pg 135):

$$q_f = 1.3\ N_c\ c + \gamma'\ DN_q + 0.6\gamma RN_\gamma \qquad 2\text{-}10a$$

where N_c, N_q and N_γ are dimensionless factors dependent only upon the angle of shearing resistance φ. For $\varphi = 0$ as for a saturated clay in undrained shear, these factors are respectively 5.7, 1 and 0 giving:

$$q = 7.4\ c + \gamma' D \qquad 2\text{-}10b$$

The total carrying capacity of the foundation would be equal to the bearing capacity times the bearing area plus the unit skin friction f_s times the pile shaft* area:

$$Q_T = Q_B + Q_s = qA + 2\pi\ R\ f_s D \qquad 2\text{-}11$$

The term γ' in equations 2-10a and 2-10b is given by the expression

$$\gamma' = \gamma + 2\,\frac{f_s + n\tau}{(n^2-1)R} \qquad 2\text{-}12$$

and is an adjusted unit soil weight to allow for the shear stress τ developed over the cylinder of radius nR, and the skin friction f_s over the pile shaft of radius R. The effect is thus to increase the surcharge unit weight γ of the soil. The value of n is chosen by trial and error to give a minimum critical load (Terzaghi, 1943, pg 135). For an incompressible soil the effect on bearing capacity of the shear on the outer boundary would be considerable; for a compressible soil such as loose sand the height over which the shearing forces τ would act would be uncertain, but much smaller.

Terzaghi provided for the effect of the compressibility of the soil by empirically reducing the cohesion and the tangent of the angle of shearing resistance to 2/3 of their test value; this effect he termed local shear failure* (Terzaghi, 1943, pg 130).

Terzaghi's analysis of the bearing capacity of a deep foundation introduced four valuable insights into pile behaviour:

1. A circular base has a bearing capacity approximately 30% higher than that of a strip footing. (The factor

TERZAGHI :

$$q = 1.3 N_c C + \gamma' D N_q + 0.6 \gamma R N_\gamma + 2D f_s / R$$

$$\gamma' = \gamma + \frac{2(f_s + \tau n)}{(n^2-1) R}$$

MEYERHOF :

$$q = N_c C + \gamma D N_q + \gamma R N_\gamma + 2D f_s / R$$

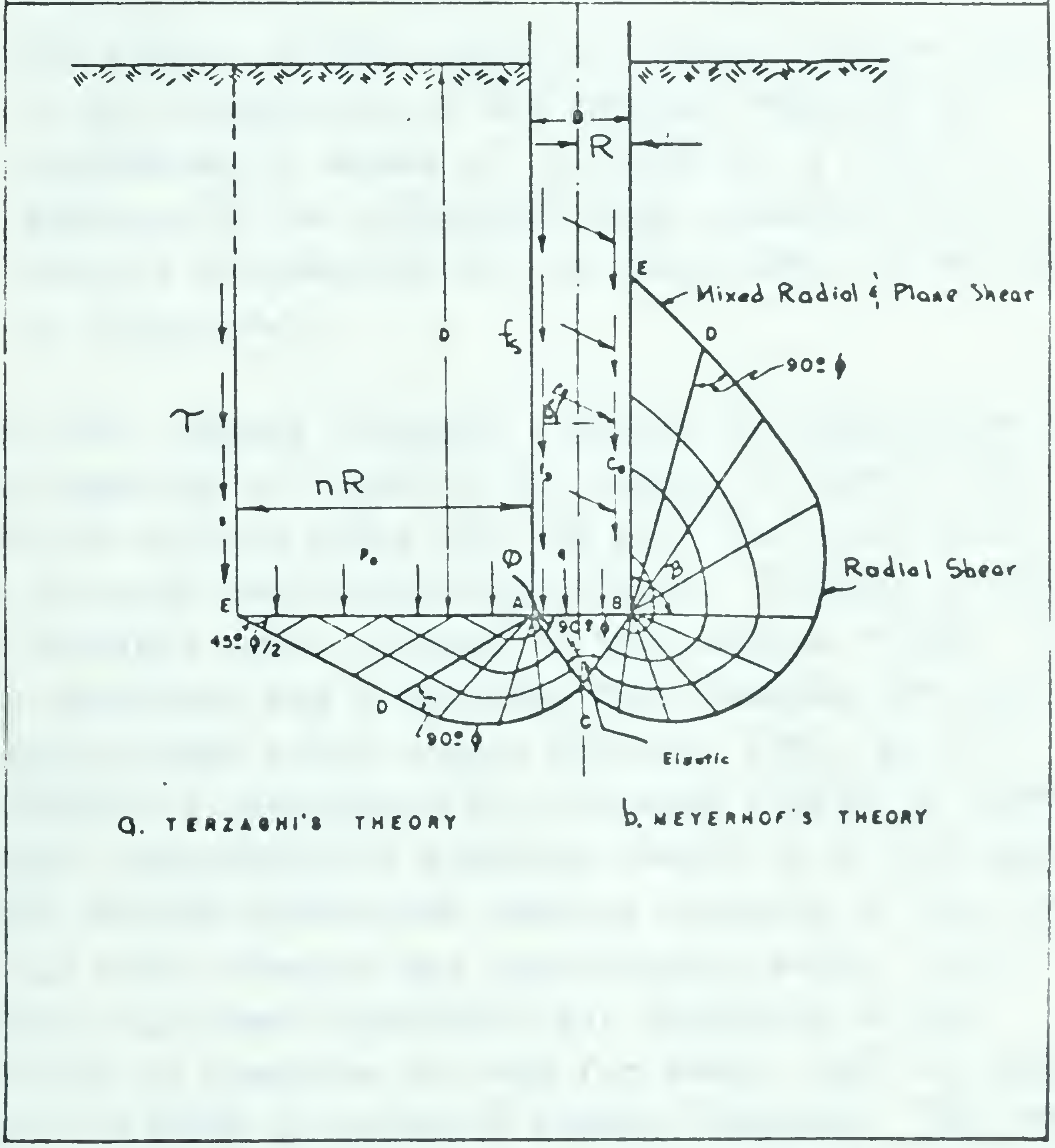


Figure 2-5 Terzaghi & Meyerhof Pile Capacity Theories

1.3 in equation 2-10a).

2. Shearing resistance can be developed in the soil above the base. The extent to which this occurs depends upon the soil compressibility (the term γ' in equations 2-10a, -10b and-11).

3. Compressibility of the soil manifests itself in local shear failure in that substantial penetration of the pile occurs well before a complete shear failure surface has developed.

4. The effect of the weight of the soil below the base of the foundation on the bearing capacity is introduced by means of the term $0.6\ \gamma\ R\ N\gamma$ of equation 2-10a calculated from consideration of the weights represented by the zones ABC, ADC and ADE of Figure 2-5.

In 1950, Wilson proposed a method of calculation of bearing capacity in terms of the theory of elasticity. He took as his failure point for the soil the yield point or stress at which the stress-strain curve diverges noticeably from a straight line, instead of the maximum stress. The failure criterion was a modified Mohr-Coulomb envelope using the defined yield stress (Wilson, 1950, pg 43). Using Mindlin's expression for stresses within an elastic, isotropic, semi-infinite material caused by a load applied at depth, Wilson determined bearing capacity at the yield point for both cohesive and cohesionless soils. His expression also made provision for variation in the coefficient of pressure at rest for sands, and for calculation of shaft friction in terms of plastic failure. The results of model and field tests on screw piles and discs were correlated with the formulae but only by using several assumptions about the coefficient of pressure at rest, Poisson's ratio and the remoulding or disturbing effect of the installation process. Although Wilson's method of

analysis met with considerable criticism, it did point out that settlement must be related to the stress-strain properties of the soil.

In 1951 two extremely important contributions to the understanding of pile behaviour were made by Meyerhof and Skempton. Meyerhof provided an extensive analysis of bearing capacity of all soil types for both shallow and deep foundations, based on plastic analysis; Skempton restricted his analysis to foundations in a saturated clay, particularly for piles, with emphasis placed on prediction of settlement under loads. Both of these contributions are largely quoted in the development of the theory in Chapter III and their work will be summarized briefly here.

Meyerhof extended the analysis of Prandtl for surface loading to the condition of deep foundations by employing the model shown in Figure 2-5 (page 15). This is an extension of the concept of Jaky (1948, pg 101) with the addition of shearing forces developed along the boundary BE of the zone BDE. The angle β may vary from $+90^{\circ}$ to -90°, representing loading conditions from a deep foundation to an unconfined compression test respectively. Meyerhof considered foundations with both smooth and rough bases, smooth and rough walls and all shapes of base. He uses the basic equation:

$$q_f = N_c\, c + \gamma D\, N_q + \gamma R\, N\gamma \qquad 2\text{-}13$$

where the terms are the same as those of Terzaghi in equation 2-10a (repeated here)

$$q_f = 1.3\, N_c\, c + \gamma'\, DN_q + 0.6\, \gamma\, R\, N\gamma \qquad 2\text{-}10a$$

but where N_c, N_q and $N\gamma$ in the Meyerhof equation are called General Bearing Factors which depend upon depth and shape of the foundation as well as the roughness of the base and φ. For deep foundations Meyerhof also makes an empirical reduction based on analysis of pile tests, for the compressibility of the soil by using:

$$c' = .9c \text{ and } \tan\varphi' = .85 \tan\varphi$$

For clays Skempton used an expression similar to that of Meyerhof and Terzaghi, but introduced a factor of safety F, and expressed the allowable bearing capacity, q_a, as

$$q_a = \frac{1}{F}\left[c \cdot N_c + p_o(N_q - 1) + \frac{\gamma B}{2} N\gamma\right] + p \qquad 2\text{-}14$$

where c = apparent cohesion of soil

p_o = effective overburden pressure at foundation level

p = total overburden pressure at foundation level

γ = unit weight of soil below foundation (submerged unit weight if foundation below water level)

B = breadth of foundation

$N_c, N_q, N\gamma$ = factors depending upon angle of shearing resistance φ, ratio of length to breadth B/L, and ratio of depth to breadth D/B.

Skempton's equation is for an excavated foundation and the total overburden pressure p is the load required to restore the soil to its initial state of stress before excavation. This is necessary since the calculation of the ultimate bearing capacity presupposes zero stress in the material (beyond that due to gravity) at the commencement of loading. Skempton thus attempts to relate the construction history to the stress condition in the soil. The purpose of the term (N_q-1) is not so clear; there

appears to be no reason why the effective overburden pressure should be used to reduce the effect of surcharge on the bearing capacity.

For purely cohesive soils, Skempton's equation 2-14 is reduced to

$$q_a = \frac{c}{F} N_c + p \qquad 2\text{-}15$$

For circular or square footings Skempton suggests that the bearing capacity factor N_{cr} is approximately 20% higher than for a strip foundation and the variation with breadth over length ratio is:

$$N_{cr}\ (\text{rectangle}) = (1+0.2\ \frac{B}{L})\ N_c\ (\text{strip}) \qquad 2\text{-}16$$

To evaluate the term N_{cr}, Skempton adapted an expression by Gibson (1950, pg 382) for the force required to expand a spherical cavity at a given depth:

$$N_{cr} = \frac{4}{3}\ (\ln \frac{E}{c} + 1) + 1 \qquad 2\text{-}17$$

where E = secant modulus of elasticity at a stress equal to $1/2\ \sigma_d$

c = cohesion intercept

ln= natural logarithm

For the usual range of E/c ratio for saturated clays, 50 to 200, the corresponding range of N_c is 7.4 to 9.4; substantially the same values found by various other methods for circular bases with depth over breadth ratios greater than 2 1/2 ie. Meyerhof 9.3 - 9.7, Rodin and Tomlinson 8.7 - 9.3, Yassin 9.7, Wilson 7.9 - 8.25 and Seed and Reese 9.0 (Insley, 1959, pg 15-16).

On the basis of elastic theory Skempton derived an approximate expression for average immediate settlement Δ of a footing at any depth:

$$\frac{\Delta}{B} = 2 \in \qquad 2\text{-}18$$

where B is the footing diameter and $\in$ is the unit axial strain in an undrained compression test at the same ratio of applied stress to failure-stress $(\frac{\sigma}{\sigma_f})$ as exists on the footing.

The work of Terzaghi, Meyerhof and Skempton laid the basis for a great deal of research on pile foundations, and many analyses of field test results. For the determination of bearing capacity based on Prandtl's mode of failure Meyerhof further investigated the effects of roughness of base and ground water conditions (Meyerhof, 1955), and Correa suggested a method of correcting for compressibility of the soil on the basis of relative density of sands and relative consistency of clays (Correa, 1960, pg 1132, 1133).

For bearing capacity as calculated on the basis of the expansion of a spherical cavity, a detailed solution based on actual stress-strain curve relationships has been proposed by Ladanyi (1963).

In Table 2-1 a brief summary is given of values of the bearing capacity factor N_c for a purely cohesive soil. These values would be applied in the form:

$$q_f = N_c \cdot c + p_o \qquad 2\text{-}19$$

where c is the undrained shear strength of the soil and p_o is the overburden pressure at the level of the base.

TABLE 2-1

SUMMARY OF BEARING FACTOR N_c FOR

PURELY COHESIVE SOIL

Source	Date	Basic Analysis	Base Shape	N_c [1]
Prandtl	1920	Plastic equilibrium	strip	5.14
Hencky	1923	Plastic equilibrium axial symmetry	circular	5.64
Hogentogler-Terzaghi	1929	Wedge zones of active and passive pressure	strip	4.0
Fellenius	1929	Circular arc	strip	5.52
Krey	1932	Circular arc and passive wedge	strip	6.05
Wilson	1941	Modified Fellenius circular arc and vertical to surface	strip	5.52
Terzaghi	1943	Plastic equilibrium,	strip	5.70
		rough base	circular	7.4
Meyerhof	1951	Plastic equilibrium,	strip	5.14
		surface, smooth base	circular	5.71
		$D/B > 2$	circular	9.4-9.7
Skempton	1951	Expanded cavity, $D/B > 2\ 1/2$	circular	7.4-9.4
Rodin	1953	Field tests on bored pile and plates	circular	8.7-9.3
Yassin	1950	Model studies $D/B > 2\ 1/2$	circular	9.7
Wilson	1950	Elastic analysis, deep foundation	circular	7.9-8.25
Seed	1957	Field tests, deep foundation	circular	9.0
Skempton	1959	Summary of pile tests	circular	9.0

1. N_c Factors are for surface loading unless otherwise stated.

2.3 Shaft Resistance

The determination of shaft resistance by analytic methods has not received the same attention as that of bearing capacity. One reason for this is that for many years most piles were driven into place, hence considerable disturbance occurred along the pile shaft and the actual condition of the soil against the shaft was unknown. Furthermore, for driven piles the dynamic pile bearing formulae as correlated with field loading tests were relied upon almost exclusively for design purposes.

The earliest attempts to evaluate pile shaft capacity were in terms of Rankine's concept of the angle of repose Early workers, Vierendel in 1907, Desmond in 1909 and Benabenq in 1911 all used expressions using the Rankine state of passive pressure as mobilized by movement of the pile (Insley, 1959, pg 3).

Subsequent workers, as typified by Dorr in 1922, introduced the concept of skin friction or friction between pile surface and soil as a function of the lateral pressure against the pile. This lateral pressure was taken as the passive earth pressure by Dorr and Caquot and Kerisel (1948), or the pressure of earth at rest by Jaky (1948, pg 100).

As more data from pile loading tests became available during the late 1940's and early 1950's it became evident to many that design values for skin friction could be determined only by load tests or by sounding tests such as the standard or cone penetrometers, and that even the results of these must be correlated empirically with load tests.

During this period the use of tables of skin friction values for design became common: Terzaghi and Peck (1948, pg 489), Chellis (1944, pg 175-181), Chellis (1951, pg 502-520) although users were warned against indiscriminate use of these values.

For soft clays the results of various tests demonstrated that in the case of rough surfaces such as rough concrete, the skin friction could be assumed equal to the shearing resistance of the soil at the pile face (Meyerhof, 1951; Peck, 1958; Tomlinson, 1957; Meyerhof, 1961). For stiff clays, arbitrarily defined as those clays having an undrained shear strength of 1000 to 1500 lbs/sq foot or more, these same authors reported that something less than the full shearing strength was developed and that a limiting skin friction or adhesion of about 2000 lbs/ sq. foot existed regardless of the shear strength of the soil. The values to be used were considered dependent upon the installation conditions of the pile. For driven piles in soft clays it was found that the adhesion immediately after driving was almost the same as the remoulded shear strength of the soil but with time it increased to or even became greater than the original shear strength of undisturbed samples due to consolidation or thixotropic effects. In stiff clays the reduced adhesion was attributed to remoulding and space between the pile and soil caused by "whipping" as the pile was driven (Tomlinson, 1957, pg 70). For cast-in-place piles the reduced adhesion was attributed to the softening effect due to excess water from the concrete or water used in the boring process (Meyerhof, 1951, pg 322; Skempton, 1959, pg 155).

A recent investigation reported in the Canadian Geotechnical Journal indicates a possibility that for some

stiff clays at least the full shearing resistance of the soil may be developed for timber piles and driven Franki piles (Lo, 1964, pg 79). For the pile tests reported, the average developed skin friction was nearly equal to the undrained shear strengths of between 1000 to 2000 lbs/ sq. ft.

Based on extensive field and laboratory testing in the stiff marine London clay, Skempton reported that the average adhesion was about 45% of the average undrained shear strength of undisturbed samples or about 80% of the average shear strength of the soil immediately adjacent to the pile at the time of pile testing (Skempton, 1959).

On the basis of the deformation properties of a pile with constant cross-section, Schenck has proposed a method of separating shaft resistance and end bearing (Schenck, 1951, pg 20-26). First formulated in 1938, Schenck's method is based on the reasoning that for a given skin friction distribution along the shaft of a pile and with a given magnitude of end bearing, the deformation of a pile at all depths is uniquely determined by the dimensions and deformation characteristics of the pile. Conversely, if the deformation of the pile at several locations is known, for example at the base, at mid-height on the shaft and at the cap*, the distribution of skin friction and the total end bearing can be determined for any given pile load.

Schenck has tabulated coefficients for some twelve different shapes of skin friction distribution. Knowing the deformations in the pile at the bottom and at approximately mid-depth, the coefficients can be used to determine the end bearing and skin friction distribution. The deformation at mid height and at the bottom can be measured by means of a steel rod inserted in a hollow sleeve in

the pile.

The last ten years have seen increased attention paid to the deformation characteristics of both pile and soil for the determination of shaft resistance. In 1955, Seed and Reese presented an analysis for a driven pile in a soft clay, relating the movement of the pile surface to the movement of a vane tip in a vane shear test and calculating the shearing resistance accordingly (Seed, 1957). This method assumes no deflection of the soil due to shearing forces in the overlying soil and can be applied to any stress-strain relationship for the soil. The end bearing of the pile is taken care of in the analysis by calculating an equivalent length of pile to give the same load capacity.

The actual distribution of load along the pile is calculated by a method of numerical integration starting with an assumed movement of the pile tip. Successive movements and loads are calculated in increments of pile depth to arrive at the total settlement and load at the top of the pile.

In 1957, Kezdi derived a similar solution to that of Seed and Reese but one which assumed an exponential stress-strain law for shear between the pile and soil. Kezdi also examined the limiting cases of free and fixed pile base. His basic differential equation for load transfer was identical with that of Seed and Reese (Kezdi, 1957, pg 47; Seed & Reese, 1957, pg 747). However, in the case where the base was free to move, he allowed for the point resistance by a semi-empirical formula and did not use an equivalent shaft length as did Seed & Reese. Although Kezdi's analysis was intended for a cohesionless soil and supported by data from a test pile in a silty-sand, there

seems little reason why the analysis could not be applied to a cohesive soil.

At the Fifth International Conference on Soil Mechanics and Foundation Engineering in Paris, Haefeli reported on a method of relating pile settlement to load by considering displacements due to shaft friction and base resistance* (Haefeli, 1961). He considered the pile as a rigid cylinder with a base of variable point angle in an elastic solid of variable depth. The settlements were calculated on the basis of Boussinesq equations which assume the soil incompressible (ie. $\mu = .5$). The skin friction was assumed to be distributed uniformly along the pile shaft causing settlements only below the pile base. The modulus of elasticity* (called "modulus of compressibility" by Haefeli) used in the resulting equations was determined in the field by a model pile; somewhat of the nature of a cone penetration test. Although many of the assumptions for the analysis could be considered unjustified, the use of a field test to determine the modulus of elasticity, and calculated on the same basis, probably eliminates many otherwise serious errors. Reasonable correlation is obtained between theoretical and actual load-settlement curves for a large scale field pile. The most serious criticism of the analysis comes from the fact that the Boussinesq equation cannot be used for problems where the load is not applied at a free surface. A similar approach by Masters (1943) for timber friction piles was criticised by Mindlin on the same basis (Mindlin, 1943, pg 147).

In an analysis which assumes the pile base fixed but includes displacements of the soil due to the overlying stress caused by the pile on the soil, D'Appolonia and Romualdi (1963a) analysed the load transferred along the

shaft of a pile in cohesionless soil. They assumed that the soil was an elastic isotropic solid with modulus of elasticity and Poisson's ratio constant with depth and that the shearing occurred within the soil, also that the coefficient of earth pressure against the shaft was that of earth pressure at rest, and numerically equal to 1.0.

With these assumptions and using Mindlin's equations for elastic settlements due to applied stresses (Mindlin, 1936) the authors arrived at a general solution in terms of the soil and pile constants in the form of simultaneous equations (D'Appolonia, 1963a, pg 17 & 18). For two field tests on steel H-piles the authors compared the actual load transfer as measured by strain gauges along the shaft with the load transfer computed by a numerical integration process from their analysis. The agreement was fairly good, with the greatest discrepencies occurring near the lower portion of the piles.

In a second paper, D'Appolonia and Hribar analysed the behaviour of a corrugated steel step-taper pile in sand and gravel, using the above mentioned analysis based on elastic theory. They found fair agreement between the theoretical and measured load transfer when allowance was made for the soil support at the step sections (D'Appolonia, 1963b).

2.4 Field Pile Tests

The general subject of field pile tests is beyond the scope of this thesis. However brief mention will be made of advances in test procedures and equipment as a background for the testing of the model pile reported herein. A comprehensive summary of current pile load tests and their evaluation is given by Fletcher (1962).

The test loading of piles has long been a requirement for foundation design and usually has consisted of a maintained load applied vertically to a pile for some period of time while settlements of the pile cap were measured. The time the load was left on was either arbitrarily set or else depended upon a minimum rate of penetration; when this rate was reached a new load was placed. Some tests were carried out using continually increasing loads; others consisted of alternate loading and unloading with measurement of settlement and rebound of the pile cap. This general type of test could be called the Maintained Load (ML) Test.

Many ingenious schemes have been devised for applying the load to the pile. One of the most common methods is that of applying a dead load on a platform; the load, called kentledge, being pig iron ingots, cement bags, scrap iron, concrete blocks etc. Another very common method is that of installing two piles adjacent to the pile to be tested and using these as reaction to a load thrust on the pile. For the latter case, and frequently for the kentledge loading, a jack is provided to apply the load to the pile. A novel source of reaction was that used by Cambefort in France in 1953. His technique was to install two steel cables adjacent to the pile and grouted into the limestone bedrock some 90 feet below the ground surface (Insley, 1959, pg 40). Because of their small size compared to the 42 inch diameter pile the two tension cables had slight effect upon the stresses in the ground, a situation which may not apply when standard piles are used for reaction. Other systems use reaction piles as a fulcrum for a loaded beam bearing on the pile, with loads provided by weights or water-filled tanks (Chellis, 1961, pg 459, 461). Where loads are not too great, reaction has been supplied by loaded trucks or low-boy trailers, or by reaction against structural members of a building (Leonards, 1962,pg 866).

For the purpose of determining the total load capacity of a pile Whitaker and Cooke in 1961 proposed a method of test whereby the pile was loaded at a constant rate of penetration of the pile cap. They called this the Constant Rate of Penetration (CRP) Test. The test was originally developed for use in model pile studies (Whitaker, 1961, pg 171) but is recommended for field tests because:

1. It shows clearly the force and penetration necessary to fully mobilize the resistance of the soil.
2. The rate of strain, suitably chosen, could approximate the conditions in the laboratory test of soil samples.
3. The test can be completed in a very short time; as little as 10 minutes (Esrig, 1963, pg 38).

Comparison with the usual Maintained Load test shows that the CRP and ML tests give essentially the same total capacity, and that the rate of penetration did not appear to affect the magnitude of the total load by more than 4% for one series of tests (Whitaker, 1961, pg 172). Because the CRP test gives no indication of settlement at other than the ultimate load, Whitaker and Cooke recommend that the CRP test be accompanied by an ML test at the design load.

Settlement of a pile is usually given in terms of vertical movement of the pile cap as measured by dial gauges, direct levelling or level readings on scales mounted on the pile cap. Dial gauges are mounted on a beam system resting on supports beyond (it is hoped) the settlement influence of the pile. Survey levels are normally referred to deep bench marks or other datum points believed to be constant. Because it is difficult to ensure a perfectly vertical and symmetrical load on the

pile, settlement readings are usually taken at diametrically opposite points on the pile top to give the settlement at the centre-line.

The magnitude of the total applied load is either calculated from the known weight of the kentledge or determined by means of a pressure cell or proving ring. Frequently the loading jack itself is calibrated to give the load as a function of the jacking pressure. Proving rings of 110 tons capacity have been used (Insley, 1959). The use of strain gauges to measure total load is reported as early as 1940 when a Wittemore mechanical strain gauge was used (Chellis, 1961, pg 464).

In the 1940's the growing need to calculate more accurately the separate contributions of shaft resistance and end bearing to pile capacity led to various ways of measuring these variables in the field. One of the most obvious was to eliminate either skin friction or end bearing. Skin friction has been eliminated by using what amounts to a plate bearing test at depth (Golder, 1954), greasing the side of the pile (Dubose, 1955, pg 158) or by separating the pile shaft from the soil by means of a sleeve. End bearing has been eliminated by the use of collapsible forms -- wood, cardboard or steel plates with shear pins of sufficient strength only to hold the pile in place during construction (Dubose, 1955, pg 158-159)(Moore, 1964, pg 34). The aforementioned methods are most easily applied to cast-in-place piles. For driven piles a design has been used consisting of a hollow shaft with the end closed by a rigid plate connected to the surface by a solid rod inside the shaft. The base could thus be loaded to failure with the shaft stationary and the shaft then loaded with the base stationary (Swart, 1958, pg 13).

Such tests, however, leave in doubt the effect of the interaction between the shaft and base stresses during loading and also the distribution of stresses along the pile shaft. The development of electrical resistance strain gauges has made it possible to measure the loads at given depths of a pile and thus the variation of load transfer with depth. Early use of this technique, using strain gauges on a pipe embedded in concrete, is reported by Crandal (1948). Recently electrical resistance strain gauges have been placed at varying depths inside steel pipe piles (Seed and Reese, 1957, pg 733) and along the flanges of steel beam sections, (D'Appolonia, 1963a).

To evaluate the base resistance of a pile, load cells have been developed to rest either on the soil or above the base over the full diameter of the pile. Early models of such cells used the principle of a fluid confined between two rigid plates roughly the diameter of the pile with a flexible gasket around the edge. A pressure gauge at the surface, properly compensated for elevation head, gives the load at the bottom of the pile (Frischmann & Fleming, 1962, pg 125).

More recently, load cells have employed electrical resistance strain gauges mounted either on horizontal beams (Dubose, 1955, pg 160) or on vertical pillars (Whitaker, 1962, pg 693) within the cylindrical load cell. Strain of the metal parts is related to stress by the modulus of elasticity of the metal used. Unfortunately, properly designed load cells are expensive and usually must be considered expendible. A recent and comprehensive load test using a 100 ton capacity load cell which was recovered after the test is described by Whitaker (1962).

2.5 Model Piles

Model piles have varied in complexity from solid, uniform cylinders of metal or wood to finely machined and instrumented devices shaped to represent belled piles.

Instrumentation of model piles is generally the same as that for field piles. Whitaker used 1/8 inch diameter brass rods in his studies of pile group settlement, measuring individual pile loads by means of small proving rings calibrated by electrical resistance strain gauges (Whitaker, 1957, pg 159). Cooke and Whitaker report on tests on single model piles with an enlarged base. The piles were made in various lengths of hollow brass tubes and with brass bases of various diameters. The total load was measured by a proving ring and the base load by means of a load cell connecting the base to the shaft. The load cell itself was a small diameter brass tube about 3 inches long with electrical resistance strain gauges around the periphery, connected by threaded fittings to the pile base (Cooke, 1961, pg 2). The shaft load was the difference between the total and base loads. For the measurement of shaft friction, electrical resistance gauges have been used both inside and outside the pile shaft (Sowers, 1961, pg 156).

2.6 Summary

The preceding survey of literature on the subject of foundation piles has revealed an increasing insight into their behaviour. The present day understanding of pile behaviour in purely cohesive soils might be briefly summed up in the following statements:

1. Below a depth/diameter ratio of approximately 1.5 the total end bearing capacity q_f at failure

of a circular base of area A_B is expressed by the relationship:

$$Q = c_B \, N_c \, A_B \qquad 2\text{-}20$$

where c_B is the average undrained shear strength below the base and N_c is a dimensionless number. N_c has been found by both plastic and elastic theory and also field measurements to be approximately 9.0.

2. The shaft resistance Q_s of surface area A_s in a soil of average undrained shear strength c is found by

$$Q_s = \alpha \, c \, A_s \qquad 2\text{-}21$$

where α is a coefficient dependent upon the consistency of the soil and the method of installation of the pile. For soft clays α approaches unity but for stiff clays seldom does so and may range from 0.3 to 0.8.

3. Skin friction or shaft resistance is mobilized at very small penetrations whereas the ultimate base resistance requires a penetration of 5% to 20% of the base diameter.

The purpose of the work reported herein was to verify some of the above points and to further develop the theory of interaction of pile and soil to cover the general cases of different soil types along the shaft and below the base. There is no know work on this subject at the present time (1964).

CHAPTER III

THEORY

3.1 Scope

The purpose of this chapter is to outline mathematical relationships by which the ultimate load capacity and settlement at the working or design load can be estimated for a single, axially-loaded pile in purely cohesive soils.

The ultimate load capacity must be known in order to evaluate the factor of safety at the working load; settlement at the design load must be known in order that total and differential settlements of the foundation are within the limits allowed for the structure carried.

3.2 Soil Properties

The soil properties of particular importance in predicting pile behaviour are shear strength, skin friction and stress-strain relationships. From a knowledge of shear strength and skin friction the load capacity of the pile can be calculated with a reasonable degree of assurance; from the stress-strain relationships the immediate settlement of the pile at the working load can at least be estimated.

The theories considered herein are developed for a soil which has, for a given placement condition and moisture content, a constant shear strength regardless of the magnitude of the normal total stress* on the failure plane. This is the property of a saturated purely cohesive

soil and is a simple case of the Coulomb shear strength equation:

$$\tau_f = c + \sigma \tan \varphi \qquad \text{3-1a}$$

with the angle of shearing resistance equal to zero. In terms of the effective stress* theory, this means that no external drainage takes place and hence, for a saturated soil, the effective normal stress on the failure plane does not increase. Stated algebraically the Coulomb equation becomes

$$\tau_f = c \qquad \text{3-1b}$$

where τ_f is the shear resistance on the failure plane at failure and c is the undrained cohesion intercept. It must be remembered that in this case the strength is determined without permitting drainage or volume change during testing. The Mohr diagram shows that in this case the shear strength is simply one half of the deviator stress*:

$$\tau_f = c = \frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_d}{2} \qquad \text{3-1c}$$

which can be easily evaluated in the unconfined compression test (Scott, 1962, pg 361). Because of its widespread use in various equations, the symbol c will be used herein to represent undrained shear strength with subscripts S and B to indicate soil around the shaft or below the base, respectively.

The foregoing assumption is valid only if drainage is either prevented or if, due to low permeability of the soil, drainage cannot occur during the loading period. For most clay soils the permeability is so low that external drainage is negligible for most testing rates. Volume change, and therefore a change in effective stress, can take place in

undrained clay soils if there is air present in the soil voids. This will result in an angle of shearing resistance as determined by undrained shear tests. The assumption of constant shearing resistance is therefore only completely valid for saturated clay soils (Skempton, 1948).

A further assumption implicit in equation 3-1b is that the magnitude of the intermediate principal stress, σ_2, has no effect upon the magnitude of the shear strength. This can introduce an error of approximately 10% in the case of plane strain* compression (Meyerhof, 1963, pg 17).

The assumption of constant shearing resistance allows many simplifications in theory and laboratory testing of soils. The assumption can be justified on the basis that many natural soils in which piles must be used are saturated clays and therefore can be analysed as purely cohesive soils.

One method of determining undrained shear strength uses the penetration of a cone into the soil. Developed initially in Sweden, the method consists of a steel cone of standard apex angle and weight W, placed with its tip on the soil surface and allowed to penetrate freely into the soil. The amount of penetration, h, is related by a semi-empirical relationship to the undrained shear strength, τ_f, of the soil. The general relationship is given by

$$\tau_f = \frac{K\ W}{h^2} \qquad 3\text{-}2$$

where K is a coefficient dependent upon the cone angle, rate of shear (and hence weight) and type of soil (Karlsson, 1961, pg 172)(Hansbo, 1957, pg 19). While an absolute value of undrained shear strength might be difficult to

assess, without establishing K empirically, relative values can easily be obtained by comparing penetration values. A rough value of K based on work by Hansbo is 1.0 (1957, pg 22), for τ_f in tons/meter^2 (metric), h in millimeters and W in grams.

The stress-strain relationships commonly used in soil mechanics are those for the uniaxial compression test obtained by plotting deviator stress, σ_d, against unit strain ϵ, the latter calculated on the basis of change in length divided by initial length. The resulting curve frequently has an initial relatively straight portion followed by a curved portion. The slope of the initial straight portion is called the initial tangent modulus of elasticity, E_T. For a curved portion the slope of a straight line joining the origin to any point on the curve defines the secant modulus of elasticity, E_s. Both modulii have dimensions of force per unit area (FL^{-2}).

In order to more conveniently plot the results from several compression tests, instead of plotting deviator stress, the ratio of this stress divided by the maximum deviator stress, $\sigma_d/(\sigma_d)max$, is plotted against unit strain (Skempton, 1951, pg 185). For undrained shear tests this is also the same as the ratio of the shear stress τ to the maximum shear stress τ_f, ie. τ/τ_f.

3.3 Skin Friction

The skin friction of a purely cohesive soil acting on a pile surface is a function of the type and consistency of the soil, the roughness of pile surface, intensity of normal stress and the method of installation (Potyondy, 1961, pg 353; Tomlinson, 1957, pg 70; Skempton, 1959, pg 154).

For soft clays (undrained shear strength less than 1000 to 1500 psf), there is some evidence that failure occurs within the soil at some small distance from the pile surface. For stiff clays it appears that failure takes place at least partly in movement between the pile and soil.

An empirical relationship between the developed maximum undrained skin friction (or unit shaft resistance) and normal total stress on a smooth surface (Potyondy, 1961 pg 351) is shown in Figure 3-1

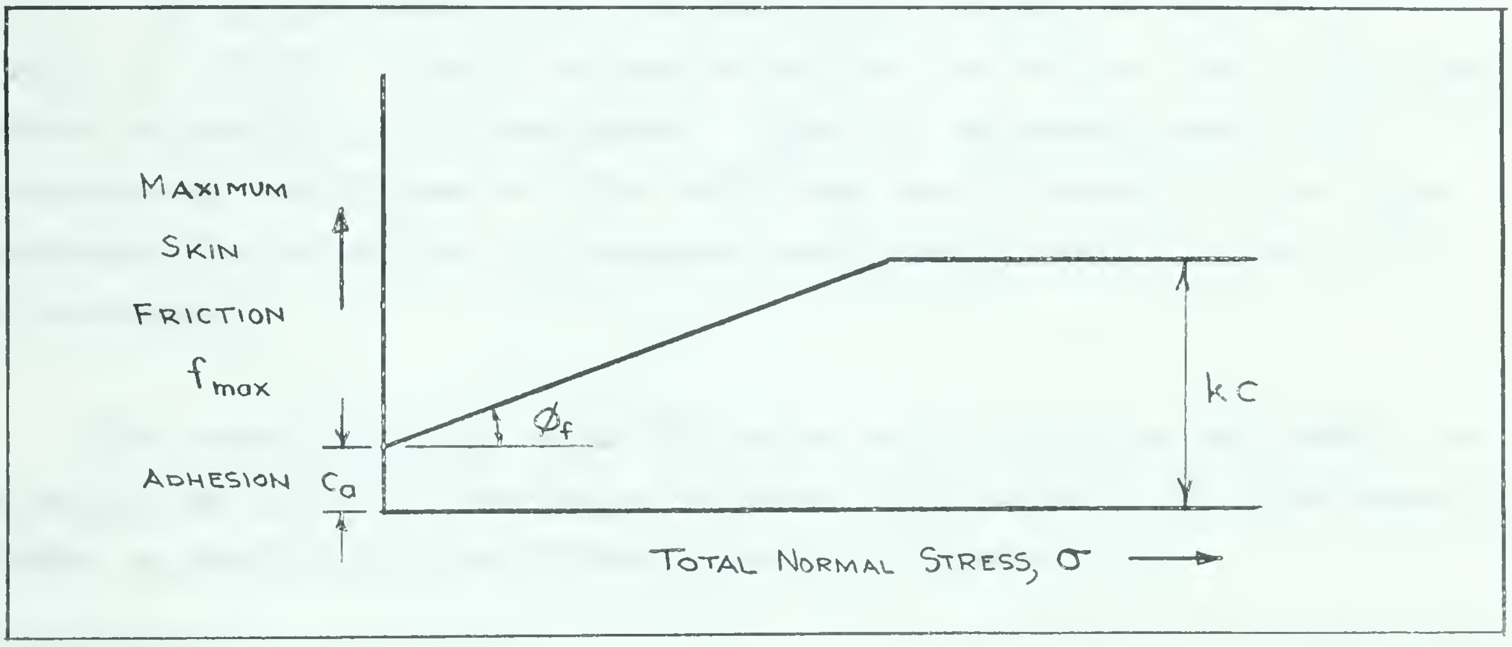


Figure 3-1 Relationship Between Maximum Skin Friction and Total Normal Stress

which stated algebraically would be:

$$f_{max} = c_a + \sigma \tan \varphi_f \circ c \qquad 3\text{-}3a$$

$$f_{max} \leq k\ c \qquad 3\text{-}3b$$

where f_{max} = maximum skin friction for given soil and surface

c_a = adhesion

σ = normal total stress

φ = angle of skin friction

c = undrained shear strength of soil

k = a constant between 0 and 1.0

This equation is almost identical in form to the Coulomb equation for the shear strength of soils. The constant k is introduced to show that for some conditions the ultimate skin friction may never increase to the shear strength of the soil, as postulated by Potyondy (1961, pg 350). Of importance here is that this equation may well apply to stiff, purely cohesive soils for which the undrained shear strength is a constant. Also it appears that with increasing stiffness of the soil and smoothness of the pile surface the adhesion c_a becomes very small and the value of k decreases.

The variation of skin friction with surface movement is similar to that of deviator stress with strain for compression tests on soil, and is illustrated in Figure 3-2.

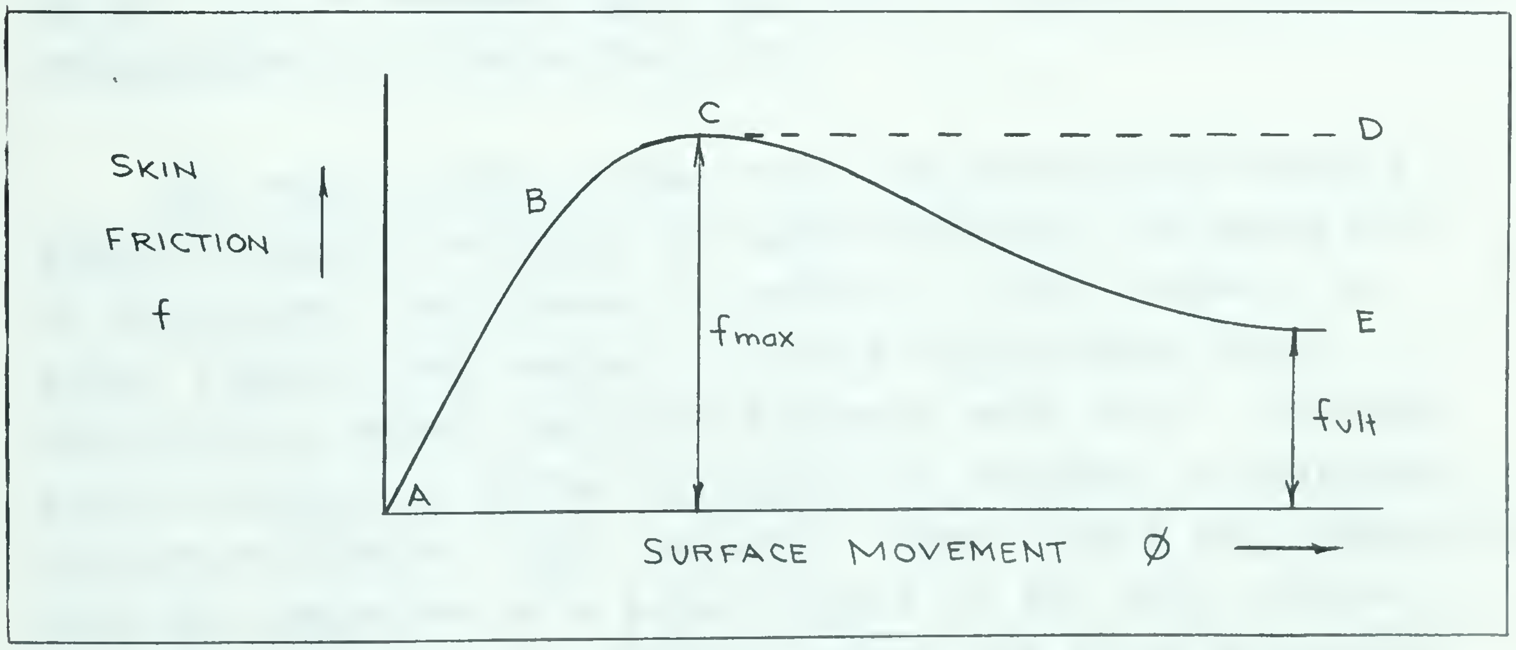


Figure 3-2 Skin Friction Stress vs. Surface Movement

The behaviour of pile skin friction in a clay soil is very complex, but in the light of the investigations made up to now (1964) it may be said to depend upon:

1. The smoothness, shape (taper, corrugations, etc.) and porosity of the solid surface.
2. The water content, degree of saturation and structure* (dispersed or flocculated) of the soil
3. The magnitude of the normal force pressing the soil against the surface, including the effect of previous normal loads.

It is now generally accepted that frictional resistance on a microscopic scale is a result of the shearing of the soil grains at the points of contact (Horn, 1962, pg 320). In 1925 Terzaghi proposed that the frictional force developed between two unlubricated surfaces was the result of molecular bonds, and that the magnitude of the force was equal to the product of the real area of contact and the unit shearing resistance of the bonds (Skempton, 1961, pg 5). It is assumed that the area of contact is directly proportional to the normal load.

The work of Horn shows that for clay-like minerals sheared against a highly polished material the magnitude of frictional resistance is reduced by the presence of polar liquids (eg. water). Horn's experiments were carried out using clay-like minerals with their cleavage surface parallel to the direction of sliding; a dispersed structure (Lambe, 1960, pg 688). Horn offers the explanation that the presence of a polar liquid at the soil surface interface reduces the adhesion since the polar molceules would become oriented to the soil surface and reduce

the bonding force between the solid surface and the soil particle. Because the adsorption of molecules is greatest along the long axis of mineral particles it would be expected that the above "lubricating" action would be greatest for dispersed soil structure and least for a random oriented (or flocculated) structure; for compacted clays this would mean that the wetter the soil is compacted the greater this lubrication action would be. Since shearing action results in an orientation of the soil particles parallel to the failure surface (Skempton, 1964, pg 81) it would be expected that as shearing progressed the magnitude of the friction resistance would be reduced; this is consistent with the stress-strain curve of Figure 3-2 which shows that the skin friction reduces after reaching a maximum value.

The effect of increased roughness of the solid surface on both a microscopic and macroscopic scale is to force the failure plane into the soil where failure becomes a function of the shearing resistance of the soil. The foregoing analysis of skin friction would therefore, not apply to a rough surface; at present (1964) no experimental criteria have been established to numerically differentiate between "rough" and "smooth".

For soils held by low normal loads against a smooth surface the relationship of Figure 3-1 shows that the maximum skin friction will not be developed. For a full-scale pile with such a surface the effect of increasing depth along the pile is to increase the normal load equal to the overburden pressure times the coefficient of earth pressure at rest (commonly taken as unity, but likely varying between 0.6 to 0.7)(Skempton, 1963, pg 273, Bishop, 1962, pg 143). This means that below a depth of say 5 to 10 feet the soil is held against the pile at a load

sufficient to develop the maximum skin friction. However for model piles or piles of shallow depth the normal load will be such as to develop only a portion of the available skin friction. For piles with a rough surface the effect of depth should not be as important except when the method of construction or subsequent shrinkage of soil is such as to prevent intimate contact between the pile surface and the soil.

3.4 Ultimate Bearing Capacity

The bearing capacity of a pile base can be estimated by any of the equations developed by Fellenius, Krey, Wilson (1941), Terzaghi (1943), Meyerhof (1951), Skempton, (1951, 1959), or Balla (1962).

The most rigorous analysis of these is that by Meyerhof first presented in 1951 and further developed by him in succeeding publications. Although only the bearing capacity of a deep circular foundation in a purely cohesive soil is of concern herein, the more general case is presented to illustrate the assumptions used in Meyerhof's method.

Meyerhof's basic premise is that the zones of plastic shear failure caused by the base, extend up above the level of the base, shown in Figure 3-3 for a strip footing. Three stress zones are created symmetrically about the centre line of the foundation:

1. A quasi-elastic wedge ABC immediately below the base, with the angle at C of approximately $90^{\circ} - \varphi$.
2. A zone of pure radial shear BCD
3. A wedge BDE of mixed radial and plane shear.

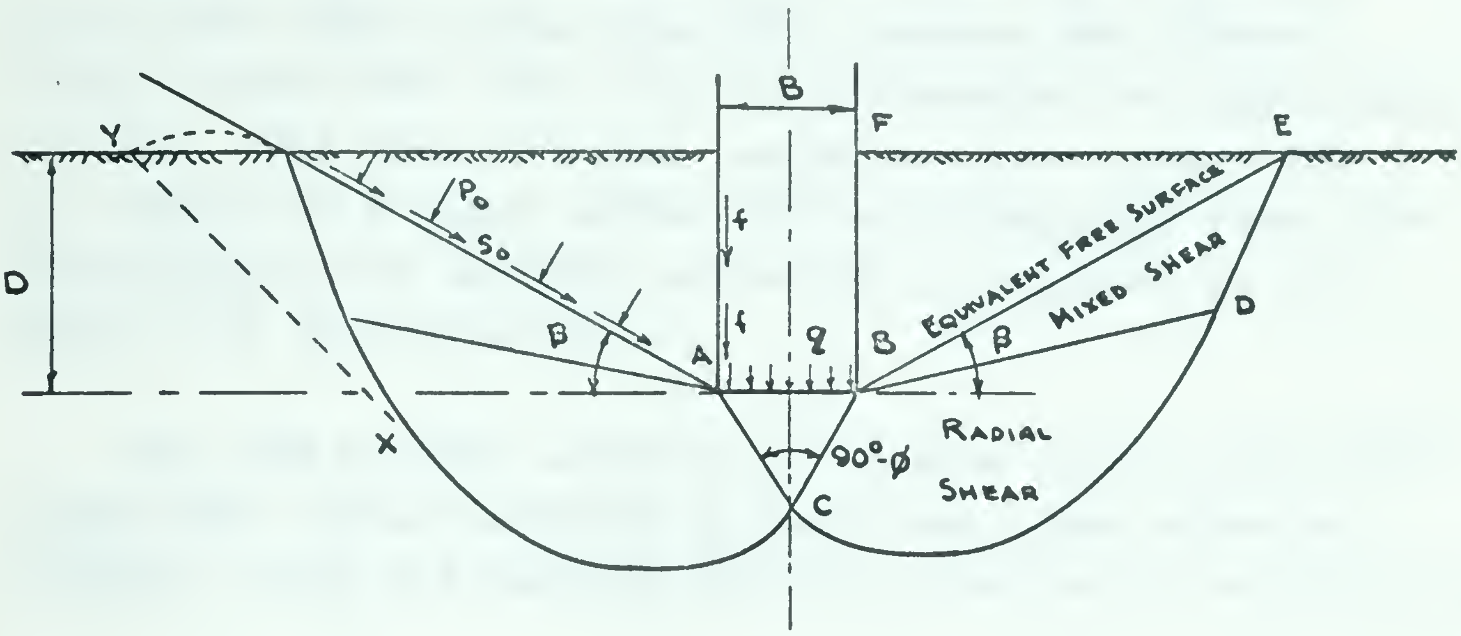


FIGURE 3-3 Plastic zones for general Meyerhof theory.

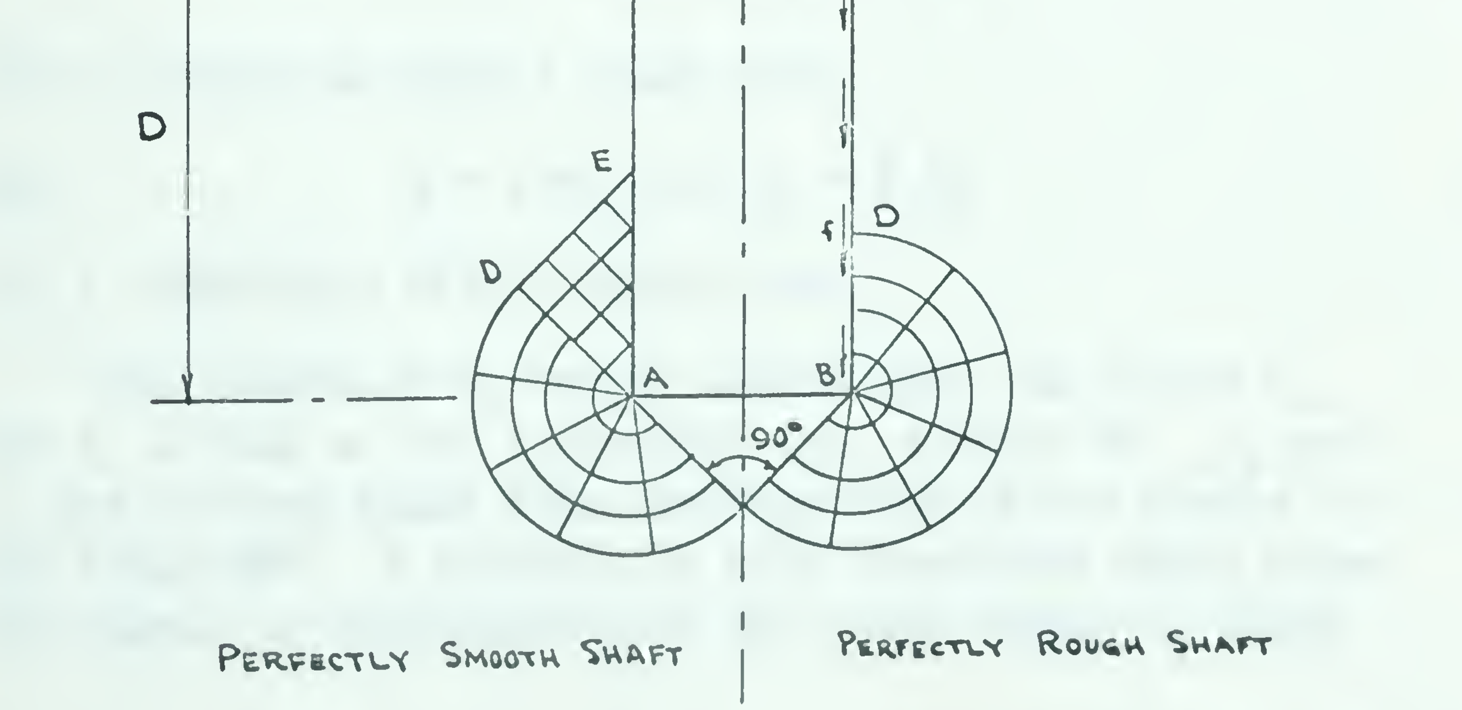


FIGURE 3-4 Plastic zones for deep foundation with rough base in purely cohesive soil.

The wedge ABC is formed as the base is loaded and forces the radial shear zone BCD downward and outward along curved paths very closely represented by logarithmic spirals. The zone of mixed radial and plane shear BDE is bounded by a plane surface BE which Meyerhof terms the "equivalent free surface" and which is inclined at an angle β to the horizontal.

For the general solution of bearing capacity Meyerhof solves the forces involved in the three shear zones by statics, using the Coulomb failure criterion of equation 3-1a:

$$\tau_f = c + \sigma \tan \varphi \qquad \text{3-1a}$$

The soil is first assumed weightless and the load q' required to cause failure is determined in terms of cohesion and angle of shearing resistance, producing the general bearing factors N_c and N_q. The soil is then assumed to have only weight and no shear strength and the load q" to balance the soil weight is found, producing the general bearing factor N_γ. The total unit load at failure q is found as the sum of q' and q", or in the final expression:

$$q = c\,N_c + p_o\,N_q + \frac{B}{2}\,N_\gamma \qquad \text{3-4}$$

for a foundation with a rough base,

and
$$q = c\,N_c + p_o\,N_q + \frac{B}{4}\,N_\gamma \qquad \text{3-5}$$

for a foundation with a smooth base.

The values of N_c and N_q depend upon the forces p_o and s_o acting on the equivalent free surface BE. p_o and s_o are in turn found from consideration of the forces on the wedge BFE. A factor m is also introduced which gives the degree of mobilization of the shear forces s_o along

the equivalent free surface ($0 < m < 1$). The value of N_γ is found with the forces p_o and s_o assumed equal to zero.

In equations 3-4 and 3-5 the general bearing factors are in terms of the angle of internal friction φ, angle of equivalent free surface β , and degree of mobilization of shear stress on the equivalent free surface m. For any given depth D, these factors must be found by determining the angle which gives the minimum bearing capacity. In addition, for cohesive soils, the possibility of failure along the plane X-Y, Figure 3-3, must also be investigated.

The value of β can vary from -90^o (which with $m=0$ represents the unconfined compression test) to $+90^o$ which represents a deep foundation; m in this case giving the degree of mobilization of skin friction in terms of shearing resistance. The intermediate case of $\beta = 0^o$ and $m=0$ represents a surface foundation.

The above is a brief description of Meyerhof's analysis for determining the general bearing factors. In the case of a deep foundation with circular base in purely cohesive soils, the stress zones, shown in Figure 3-4, are of the same general nature as those of Figure 3-3 but are smaller in size and are projected on vertical radial planes. The problem for a circular base becomes one of axially symmetrical strain rather than plane strain, as in the case of the strip footing, since strain will take place in three directions in the shear zones.

Using an analysis similar to that for a strip footing, but allowing for the radial stresses set up under a circular base, Meyerhof determines a simple formula for bearing capacity in a cohesive soil, in terms of a bearing

capacity factor N_{cqr}, k_o the coefficient of earth pressure at rest ($k_o \doteq 1$), and the overburden pressure $D\,\gamma$:

$$q = c\,N_{cqr} + k_o\,D\,\gamma \tag{3-6a}$$

or with k_o taken as unity

$$q = c\,N_{cqr} + D\,\gamma \tag{3-6b}$$

N_{cqr} has the values shown in Table 3-1, depending upon the degree of roughness of shaft and base.

TABLE 3-1

VALUES OF N_{cqr} FOR SMOOTH AND ROUGH BASE AND SHAFT

Depth	Base (Circular or Square)			
	Smooth		Rough	
Surface	5.71		6.18	
Deep Foundation $D > 2B$	Smooth Shaft	Rough Shaft	Smooth Shaft	Rough Shaft
	8.8	9.3	9.34	9.74

The foregoing analysis is based on the behaviour of a perfect rigid plastic material, that is, failure takes place over all sections of the stressed zones at the same time, and presupposes an incompressible soil. For saturated clays the assumption of incompressibility is probably justified (Skempton, 1951, pg 184).

3.5 Shaft Capacity

For purely cohesive soils most authorities calculate the shaft capacity* Q_s, using the product of an average

skin friction f and the shaft area, A_s thus

$$Q_s = f\ A_s \tag{3-7a}$$

The skin friction is variously taken as equal to the undrained shear strength of the soil adjacent to the shaft, the fully softened undrained shear strength (Meyerhof, 1951, pg 323), or some fraction of the undrained shear strength depending upon the soil consistency (Tomlinson, 1957, pg 67). Using this concept, the shaft capacity can be expressed as:

$$Q_s = \alpha\, c_u\, A_s \tag{3-7b}$$

where α is the ratio of average skin friction developed to undrained shear strength. (Skempton, 1959, pg 154).

The use of an average skin friction value presumes that the undrained shear strength of the soil is uniform over the pile depth, and the pile penetrates as a rigid body so that all parts of the pile reach the maximum movement required to develop the maximum skin friction f_{max} as shown in Figure 3-2. This failure is similar to the perfect rigid plastic failure assumed in the derivation of the bearing capacity formula.

<u>3.6 Load Capacity</u>

The load capacity Q of a pile is found as the sum of the bearing capacity Q_B and shaft capacity Q_s less the weight of the pile W;

$$Q = Q_B + Q_s - W \tag{3-8a}$$

or in terms of the statics of a pile

$$Q = c_s\ N_c + \gamma D\ \ A_b + \ \alpha\, c_B A_s - W \tag{3-8b}$$

If the weight of the pile is almost equal to the term $\gamma D A_b$, which is frequently the case, (Skempton, 1959, pg 154), then

$$Q = c_B N_c A_B + \alpha c_s A_s \tag{3-8c}$$

This formula implies that the failure load on the pile occurs when the bearing capacity and shaft capacity are each at their maximum value, but ignores the fact that to be so they must both occur at the same penetration of the pile. In fact this is not always the case, as has been demonstrated for both model piles (Cooke, 1961, pg 5) and field piles (Frischmann, 1962, pg 130), with the maximum shaft capacity occurring at much lower penetration than the bearing capacity. Whether or not the actual value of shaft resistance is less at the maximum bearing capacity depends upon the shape of the skin friction-pile movement curve shown in Figure 3-2 page 39. For behaviour shown by the curve ABCE the resistance at the failure of the base in bearing might be half of its maximum value. Therefore the coefficient α in equation 3-8b and 3-8c should be based on the ultimate skin friction rather than the maximum skin friction.

3.7 Settlement of Pile

The theory developed so far for pile behaviour has concerned itself with plastic failure of the soil by the pile load. To the load capacity so determined would be applied a factor of safety, usually about three, to arrive at a design load (Meyerhof, 1951, pg 301; Skempton, 1951, pg 180). The structural engineer would be content with this design load only if it would result in settlements which would not damage the proposed structure. It is

therefore desirable to have some method of estimating the settlement which will occur under the working load of a pile. The settlement of concern to the structural engineer will include both the immediate, consolidation and secondary compression settlements and deep-seated settlement. The analysis herein is concerned only with the immediate settlement due to the stresses created by the pile load assuming no volume change of the soil mass surrounding the pile. Settlements due to consolidation and secondary compression must take into account not only the stresses imposed on the soil by the single pile but must also consider the effect of loads imposed by the entire structure.

The following load-settlement analysis is given as a basis for estimating the immediate or elastic settlement of a pile in a purely cohesive soil. The analysis, while general in terms of soil properties, is considered especially applicable to large-diameter belled piles for which elastic settlements are of practical significance. The analysis is based largely on the work of N.B. Hobbs (1963) and detailed references will not be made to his methods.

The end bearing capacity q of a deep circular foundation is given by equation 3-6b (with k_o equal to 1.0)

$$q = c\,N_{cqr} + D\,\gamma \tag{3-6b}$$

The value of q is the net base pressure to cause failure. It is therefore necessary to take into account the initial stress condition at the base of the pile before loading.

Referring to Figure 3-5, it can be seen that the stress in the soil before excavation for the pile would be

$$\sigma_z = \gamma\,D$$

Excavation would reduce this stress to zero over the pile base area A_B and would result in an average heave of ρ_h. Placing the pile would again load the soil and if no arching occurred along the shaft and the unit weight of concrete is taken as equal to that of the soil, then the initial heave would be removed. For typical unit weights of concrete and soil the error in load on the soil might be 10% to 20% of the pile weight; in practice this would likely be offset by arching along the pile shaft. For simplicity in analysis it will be assumed that no net heave has occurred under the pile base and that no shearing stresses exist along the pile shaft.

The average net pressure q on the pile base under a load Q_T will be, from Figure 3-5:

$$q = \frac{Q_T - Q_s}{A_B} \qquad 3\text{-}9$$

where Q_s is the total shaft resistance. By the theory of elasticity the immediate settlement Δ_B of a rigid circular footing of width B under a unit pressure q will be (Terzaghi, 1943, pg 399):

$$\Delta_B = q\ B\ I\ \frac{(1-\mu^2)}{E} \qquad 3\text{-}10$$

where I = influence value for settlement

μ = Poisson's ratio

E = secant modulus of elasticity for the soil

The value for I depends upon the rigidity and shape of the base and the depth of the footing below the surface (Mindlin, 1936, pg 203). For footings of various shapes Fox has related the settlement of a footing depth D to that of a footing at the surface, both for a semi-infinite

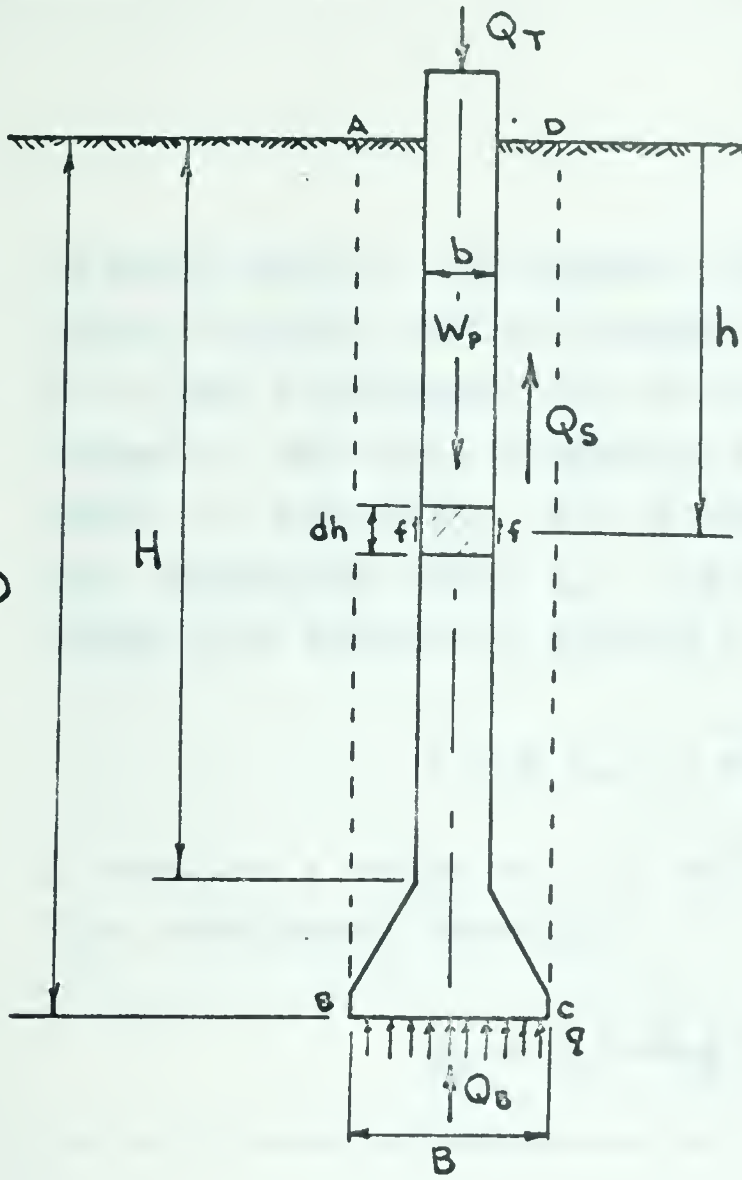


Figure 3-5 Pile loads and dimensions

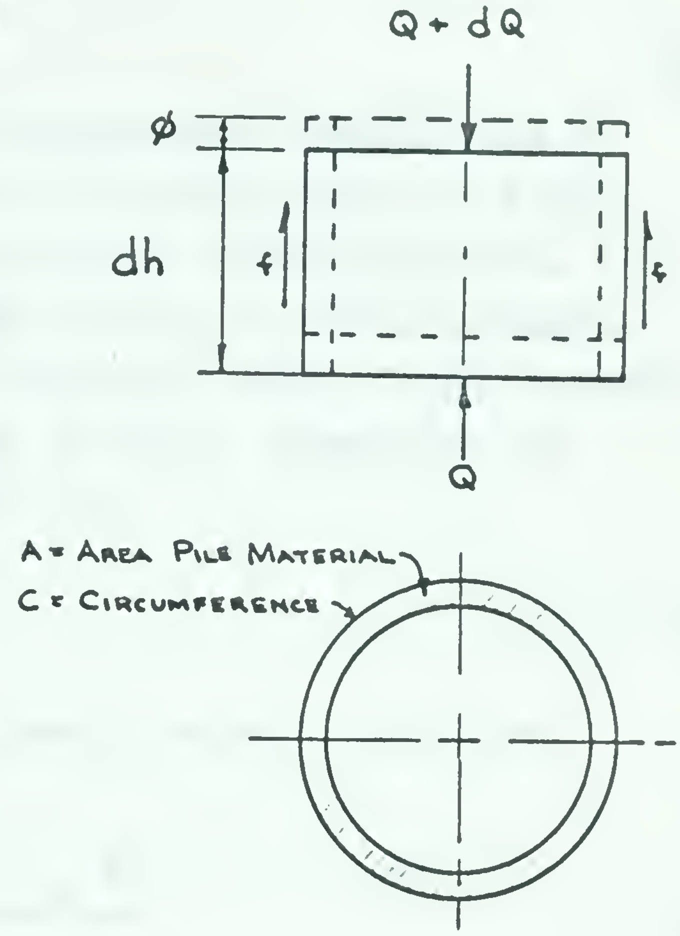


Figure 3-6 Element from pile shaft

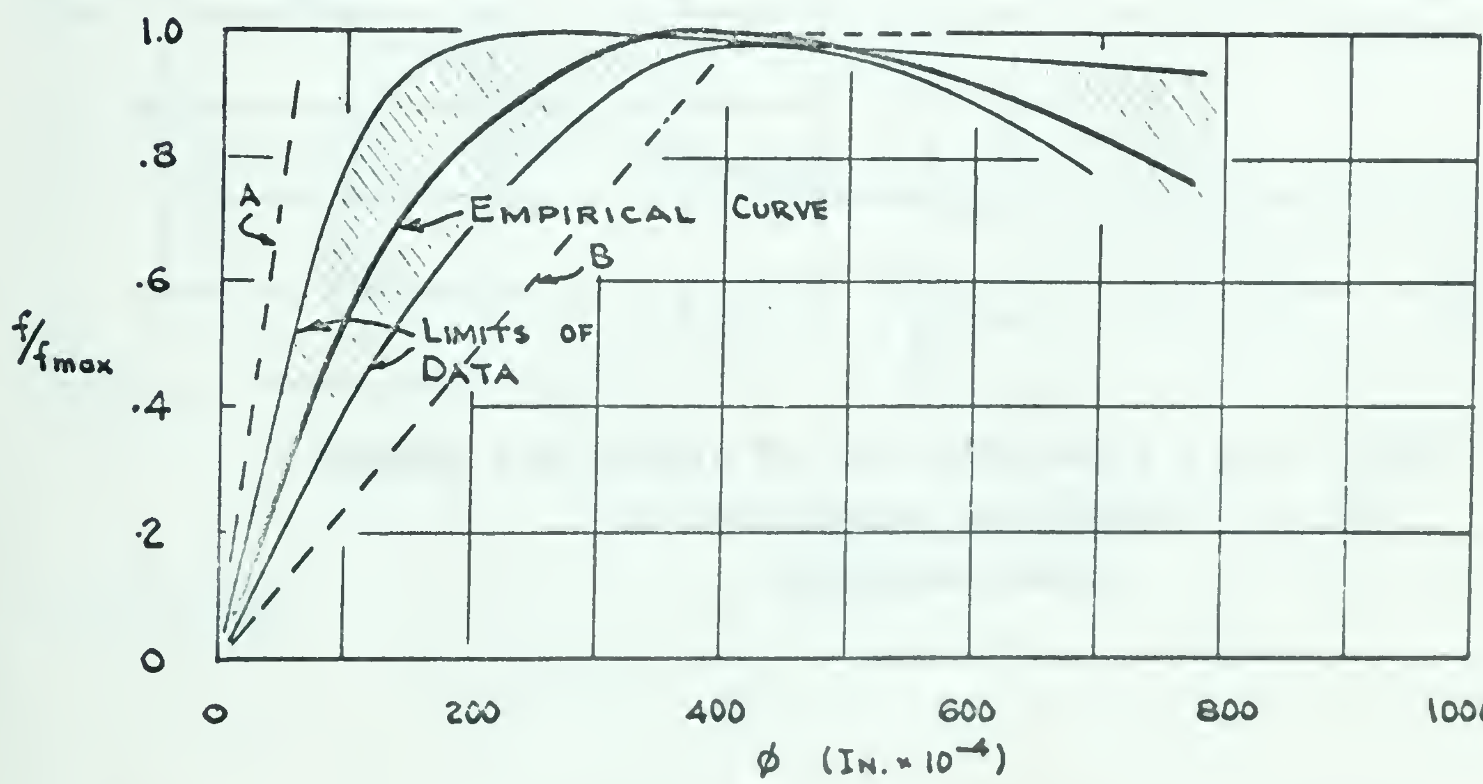


Figure 3-7 Pile movement, Ø, vs. Relative skin friction f/f_{max}

elastic solid. In Figure 3-8 is shown the coefficient M which relates the settlement for a circular base at depth D to the settlement for a circular base at the surface, in terms of the base diameter B with Poisson's ratio assumed equal to one-half. For a rigid circular base at the surface the influence value I_∞ is equal to $\pi/4$. Therefore at depth the influence factor I will be

$$I = M\ I_\infty = \frac{\pi M}{4} \qquad \text{3-11}$$

M reaches a value of 0.5 as the depth becomes infinite. The settlement thus is:

$$\Delta_B = q\ \frac{B\ M\ I_\infty\ (1-\mu^2)}{E} \qquad \text{3-12}$$

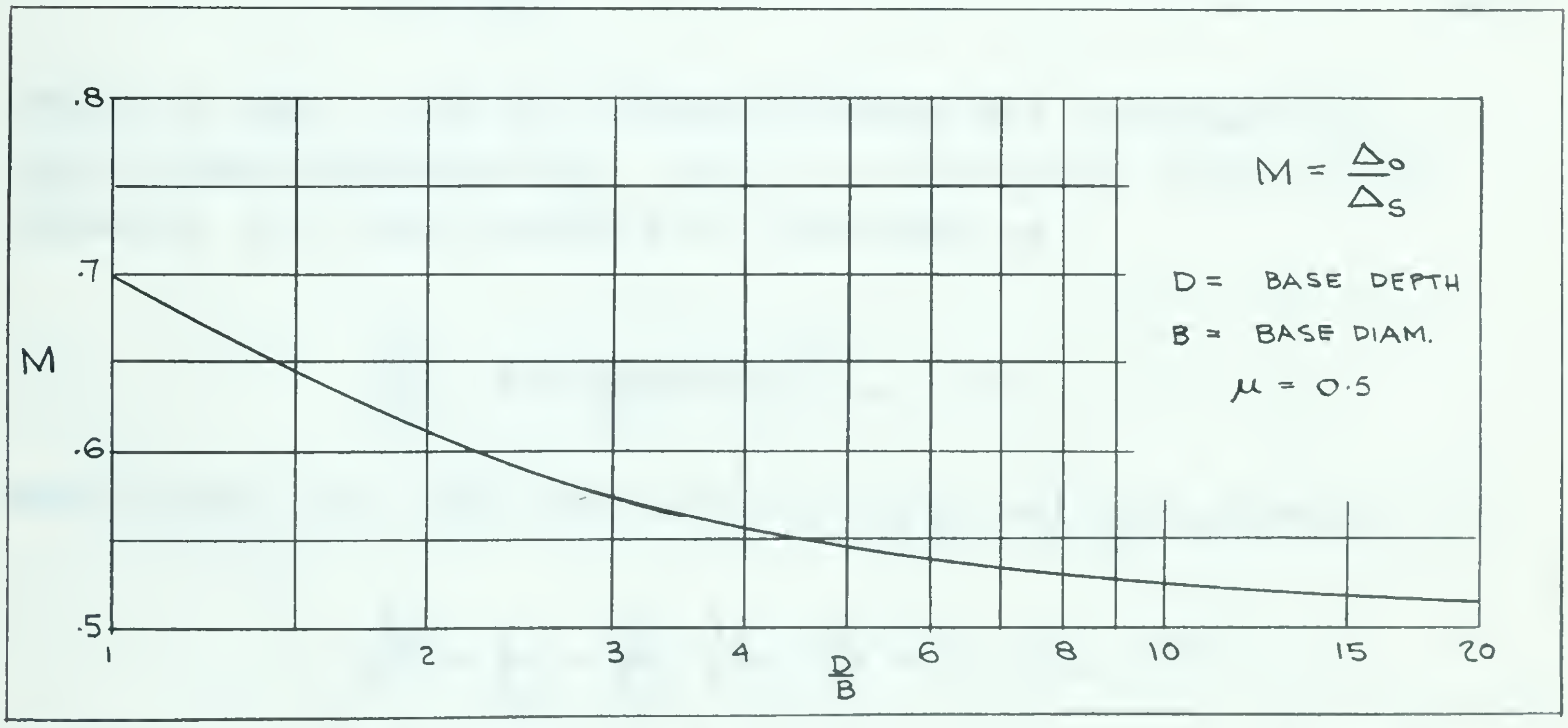


Figure 3-8 Ratio M, of Settlement Δ_o, at Depth D to Settlement at Surface, Δ_s for Circular Base

Using the same method by which Skempton produced equation 2-18, page 20, it is possible to derive an approximate formula representing the settlement of the base of a pile at a depth D, for any ratio of base load to failure base load q/q_f. The relationship is based upon the theory that the settlement of equation 3-12 will be given correctly for any value of q up to the failure pressure q_f provided that the secant modulus of elasticity, corresponding to the same ratio of deviator stress to failure deviator stress, σ/σ_d, is used.

For the axial compression test the secant modulus of elasticity E is given by Skempton (1951, pg 184):

$$E = \frac{\sigma_d}{\epsilon} \qquad 3\text{-}13$$

where σ_d and ϵ are the deviator stress and corresponding unit strain respectively. Also, for saturated clays $\sigma_d = 2\tau$. Equation 3-12 may therefore be rewritten as

$$\frac{\Delta_B}{B} = \frac{q\ M\ I_\infty (1-\mu^2)}{2\tau} \cdot \epsilon$$

Multiplying the right hand side by q_f/q_f and τ_f/τ_f gives

$$\frac{\Delta_B}{B} = \frac{q}{q_f} \circ \frac{q_f}{\tau_f} \circ \frac{\tau_f}{\tau} \circ \frac{\epsilon}{2}\ M\ I_\infty\ (1-\mu^2)$$

However $\qquad N_c = \dfrac{q_f}{\tau_f}$

Therefore $\qquad \dfrac{\Delta_B}{B} = \left[\dfrac{q}{q_f} \circ \dfrac{\tau_f}{\tau}\right] \left[\dfrac{N_c M\ I_\infty}{2}\ (1-\mu^2)\right] \epsilon \qquad 3\text{-}14$

For equal ratios of q/q_f and τ/τ_f and corresponding unit strain ϵ equation 3-14 reduces to

$$\frac{\Delta_B}{B} = \frac{N_c \, M \, I_\infty \, (1 - \mu^2)}{2} \cdot \epsilon \qquad 3\text{-}15$$

and the load settlement curve is calculated accordingly. The value of N_c can be either some average value from theory or field tests such as 9.0 or can be calculated from equation 2-17:

$$N_{cr} = \frac{4}{3} \left(\ln \frac{E}{c} + 1 \right) + 1 \qquad 2\text{-}17$$

where $N_{cr} = N_c$ and $c = \tau_f$.

The above equations provide a basis for predicting the settlement of the base of a pile; in order to determine the settlement of the pile cap the effect of the pile shaft must be introduced.

Considering an elemental section of a loaded pile of cross section area A_p and circumference C as shown in Figure 3-6, a differential equation relating load distribution and deformation of pile and soil can be approximated on the assumption that the soil adjacent to the pile deforms in accordance with a known relationship; e.g. Figure 3-7, but is not affected by the distribution of stress from loads above the point considered (Seed, 1957, pg 747).

The movement ϕ of the pile wall at depth h differs from that at h + dh by an amount $d\phi$ equal to the compression of the element dh by the load dQ. Thus, by Hooke's law*

$$\frac{d\phi}{dh} = \frac{Q}{E_p \, A_p} \qquad 3\text{-}16$$

in which E_p is the modulus of elasticity of the pile. Then,

$$Q = E_p A_p \frac{d\phi}{dh} \qquad 3\text{-}17$$

and

$$\frac{dQ}{dh} = E_p A_p \frac{d^2\phi}{dh^2} \qquad 3\text{-}18a$$

Summing the vertical forces on the element dh of Figure 3-6 gives

$$dQ = f \circ C \circ dh \qquad 3\text{-}18b$$

or

$$\frac{dQ}{dh} = f \circ C \qquad 3\text{-}18b$$

where f is the skin friction and C is the circumference of the pile. Hence

$$\frac{d^2\phi}{dh^2} - \frac{C}{E_p A_p} \circ f = 0 \qquad 3\text{-}19$$

If the surface movement ϕ could be uniquely related to the skin friction f, such as by the curves of Figure 3-7, then equation 3-19 could be solved rigorously within the frameworkof the assumptions made. For curves A and B in Figure 3-7 the solution is relatively simple; for curve A, equation 3-19 becomes:

$$\frac{d^2\phi}{dh^2} - \frac{C}{E_p A_p} \circ \psi f/f_{max} = 0 \qquad 3\text{-}20$$

where ψ is the slope of curve A such that $f = \psi f/f_{max}\phi$ (Seed, 1957, pg 747). However the usual type of deformation curve for surface movement of vane shear tests or skin friction tests is of the shape of the curves ABCD or ABCE of Figure 3-2 (Seed, 1957, pg 740; Hurtubise, 1961, pg 76).

An empirical curve which fits the data for some direct shear tests on brass plates for test A-4 in the model pile tests reported herein is given in Figure 3-7. The equation of the curve has the general form[1].

$$f = f_{max} \cdot \frac{\phi}{\phi_f} \cdot e^{1-\phi/\phi_f} \qquad 3\text{-}21$$

where f = skin friction at surface movement ϕ

ϕ_f = surface movement at failure

f_{max} = skin friction at failure

For this expression equation 3-19 becomes

$$\frac{d^2\phi}{dh^2} - \frac{C}{E_p A_p \phi_f} \cdot \phi\, e^{1-\phi/\phi_f} = 0 \qquad 3\text{-}22$$

The solution of this equation with suitable boundary conditions would provide a unique expression for the behaviour of any pile for which equation 3-21 is valid.

For a pile under a load Q_T the boundary conditions for equation 3-22 are:

1. When h=0, $\phi_o = \Delta_o$ (the settlement at the top of the pile) and $\frac{d\phi}{dh} = \frac{Q_T}{E_p A_p}$
2. When h=H (the depth to bottom of the shaft and top of the base)

---

1. Other useful equations are : $f = f_{max}\left[\frac{\phi}{a+b\phi}\right]$, $f = f_{max}\sqrt{\frac{\phi}{a-b\phi}}$, or $f = f_{max}\frac{\sqrt{\phi}}{a-b\phi}$, where a is the initial cotangent and 1/b is the maximum value of f (Konder, 1963, pg 120).

$$\frac{d\phi}{dh} = \frac{Q_B}{E_p A_p} \text{ and } \phi_H = \Delta_B \text{ (the settlement of the base),}$$

or from equation 3-15

$$\phi_H = \frac{N_c B\ M\ I_{\infty}\ (1-\mu^2)}{2} \cdot \epsilon \qquad 3\text{-}23$$

The rigorous solution of equation 3-22 for the general boundary conditions is extremely difficult; the problem is probably best solved by using numerical integration (Seed, 1957, pg 747, 763). Kezdi has solved the same basic differential equation for the cases of a pile with a fixed end (Δ_B=0) and free end (Q_B= 0) in a cohesionless soil (Kezdi, 1957, pg 46, 47). Seed solved the same equation by numerical procedure but accounted for the base resistance by adding an equivalent length to the shaft depth (Seed, 1957, pg 748).

Despite the difficulties attending the use of equation 3-22, it does demonstrate the interplay of the compression of the pile shaft, the transfer of part of the pile load to the soil and the deformation of the base soil by the pile tip under the balance of the pile load. The total settlement of the pile cap is thus the sum of the elastic shortening of the pile shaft and the penetration of the pile base.

3.8 Effects of Mould Boundaries

One of the most serious drawbacks to model analysis of any kind is the effect upon the model of boundary conditions imposed by the test equipment. Of particular concern in the model pile tests was the proximity of the rigid walls and base of the mould.

On the basis of elastic theory it is possible to estimate the distribution of vertical stresses caused by shaft and base loads of a pile. Using an analysis of Mindlin (1936) for stresses due to a point load at some depth in a semi-infinite body, Burmister arrived at a graphical representation of vertical stresses due to pile loads (1940, pg 339). The assumptions in the analysis are:

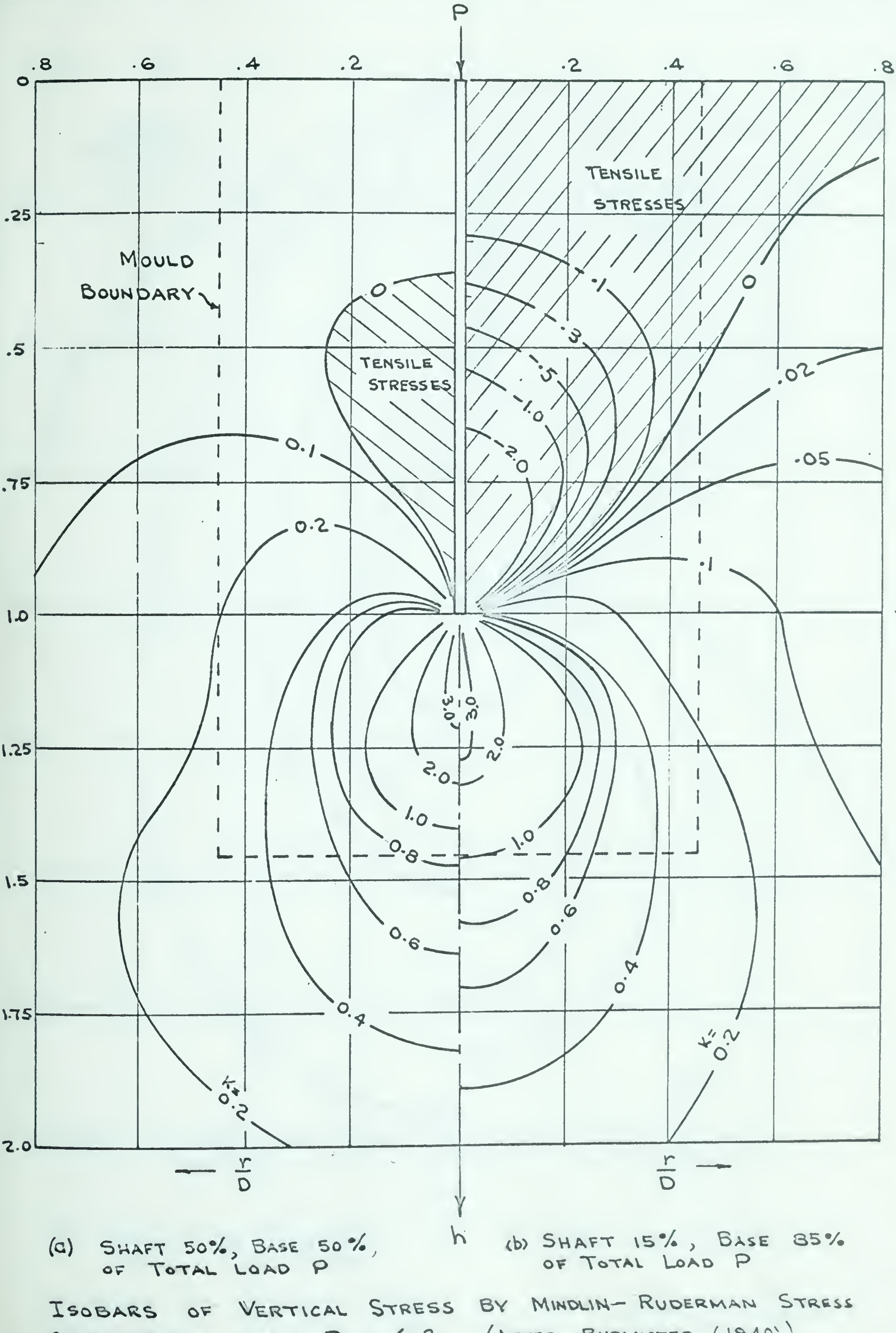
1. The pile is in a seim-infinite elastic solid,
2. Poisson's ratio of the soil is 0.5,
3. The soil is weightless and can withstand tensile stresses,
4. The shaft load is distributed uniformly and acts along the centre line of the pile,
5. Yielding at stress concentrations at the base does not alter the stress distribution.

The resulting curves representing a coefficient proportional to vertical stress for the condition of total load carried by the shaft or total load carried by the base are shown in Figures 3-9a and 3-9b respectively. The actual vertical stress σ_h at any depth h and radius r, from centre of the pile is given by the equation

$$\sigma_h = \frac{P \cdot k}{D^2} \qquad 3\text{-}24$$

where p = the total load carried by the pile
k = the coefficient found from the graph at a depth z/D and radius r/D
D = depth from surface to base

By assuming that stresses due to the base and shaft loads can be added by superposition, Figure 3-10 has been prepared for cases intermediate between those shown in Figure 3-9. Figure 3-10a shows vertical stresses for the

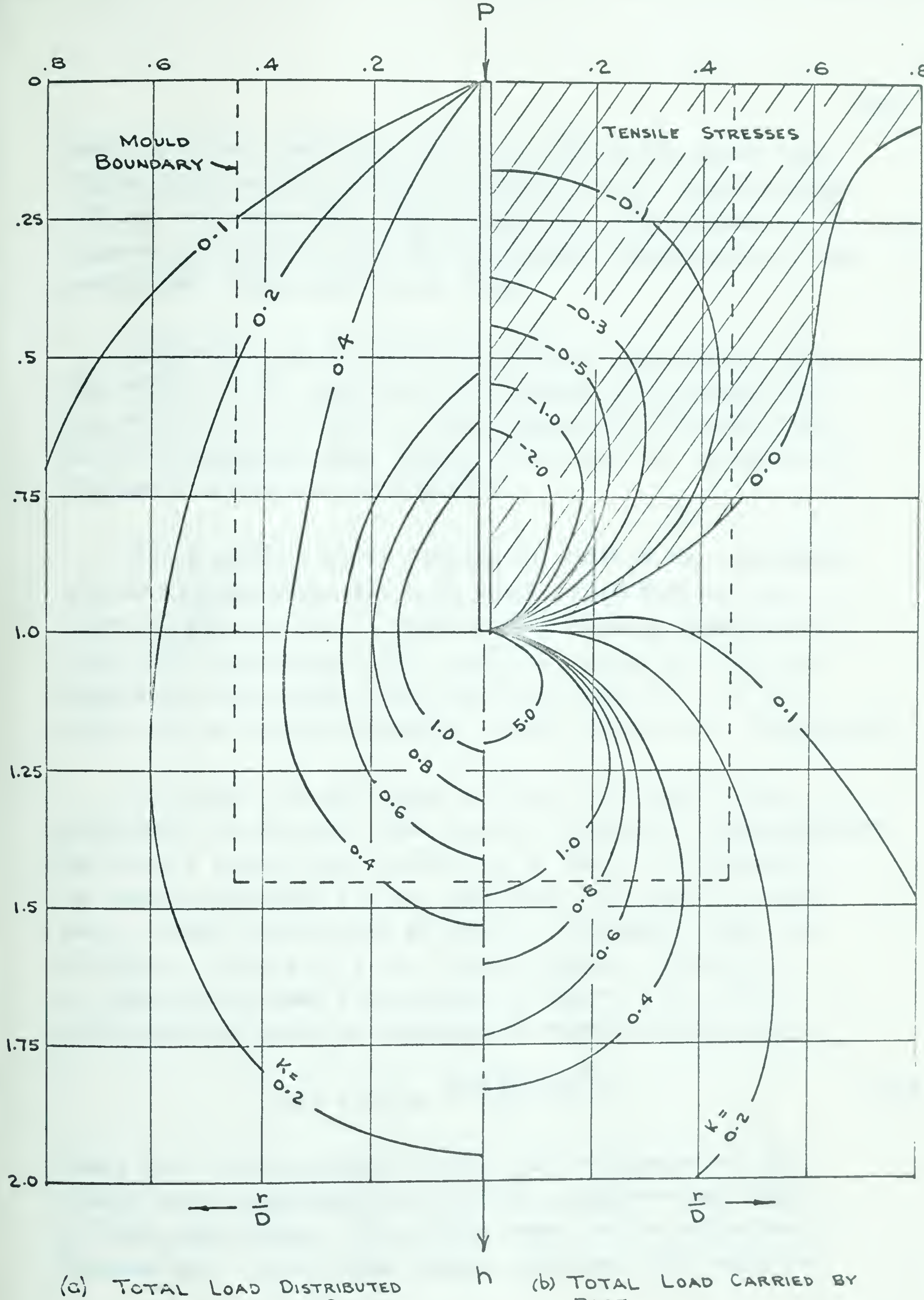


(a) SHAFT 50%, BASE 50%, OF TOTAL LOAD P

(b) SHAFT 15%, BASE 85% OF TOTAL LOAD P

ISOBARS OF VERTICAL STRESS BY MINDLIN–RUDERMAN STRESS COEFFICIENTS. $\sigma_h = P \cdot K / D^2$ (AFTER BURMISTER (1940))

Figure 3-9



(c) TOTAL LOAD DISTRIBUTED UNIFORMLY ALONG SHAFT

(b) TOTAL LOAD CARRIED BY BASE

ISOBARS OF VERTICAL STRESS BY MINDLIN-RUDERMAN STRESS COEFFICIENTS. $\sigma_h = P \cdot K / D^2$ (AFTER BURMISTER (1940))

Figure 3-10

shaft and base loads each equal to 50% of the total load. Figure 3-10b shows vertical stresses for the shaft taking 15% and the base 85% of the total load. Also shown on Figures 3-9 and 3-10 are the approximate boundaries of the mould used in the model pile tests.

Inspection of the vertical stress curves shows that the effect of the mould wall and bottom are probably not too serious in the case of loads taken by the shaft only but with increased load taken by the base the stresses at the mould bottom are substantial.

For a surface strip footing of width B the increased stress concentration due to a rough rigid surface at a depth of $\frac{B}{8}$ results in a reduction in bearing capacity of about 22% (Jurgenson, 1934, pg 153). Since for the pile dimensions the actual depth would be from 2B to 4B the reduction in bearing capacity is not likely to be significant.

The effect of the rigid bottom of the mould upon settlement of the pile base under a load can be approximated by using a correction coefficient g which is determined by elastic analysis for the condition of a rigid footing over a rough rigid layer at depth z (Terzaghi, 1943, pg 423-424). Values of g for varying ratios of depth z, to diameter of base B are given in Figure 3-11. The settlement is given by multiplying equation 3-13 by g:

$$\Delta_B = g\ M\ I_{\infty}\ \frac{q\ B\ (1-\mu^2)}{E} \qquad 3\text{-}25$$

where I_{∞} = the influence factor for settlement of the loaded area (depending on rigidity, roughness and shape of the loaded base). The other terms are as explained in Section 3-7. On the same basis, equation 3-15 could be

be modified for the case of a rough base to give:

$$\frac{\Delta_B}{B} = \frac{N_c\ g\ M\ I_\infty\ (1-\mu^2)}{2} \cdot \epsilon \qquad 3\text{-}26$$

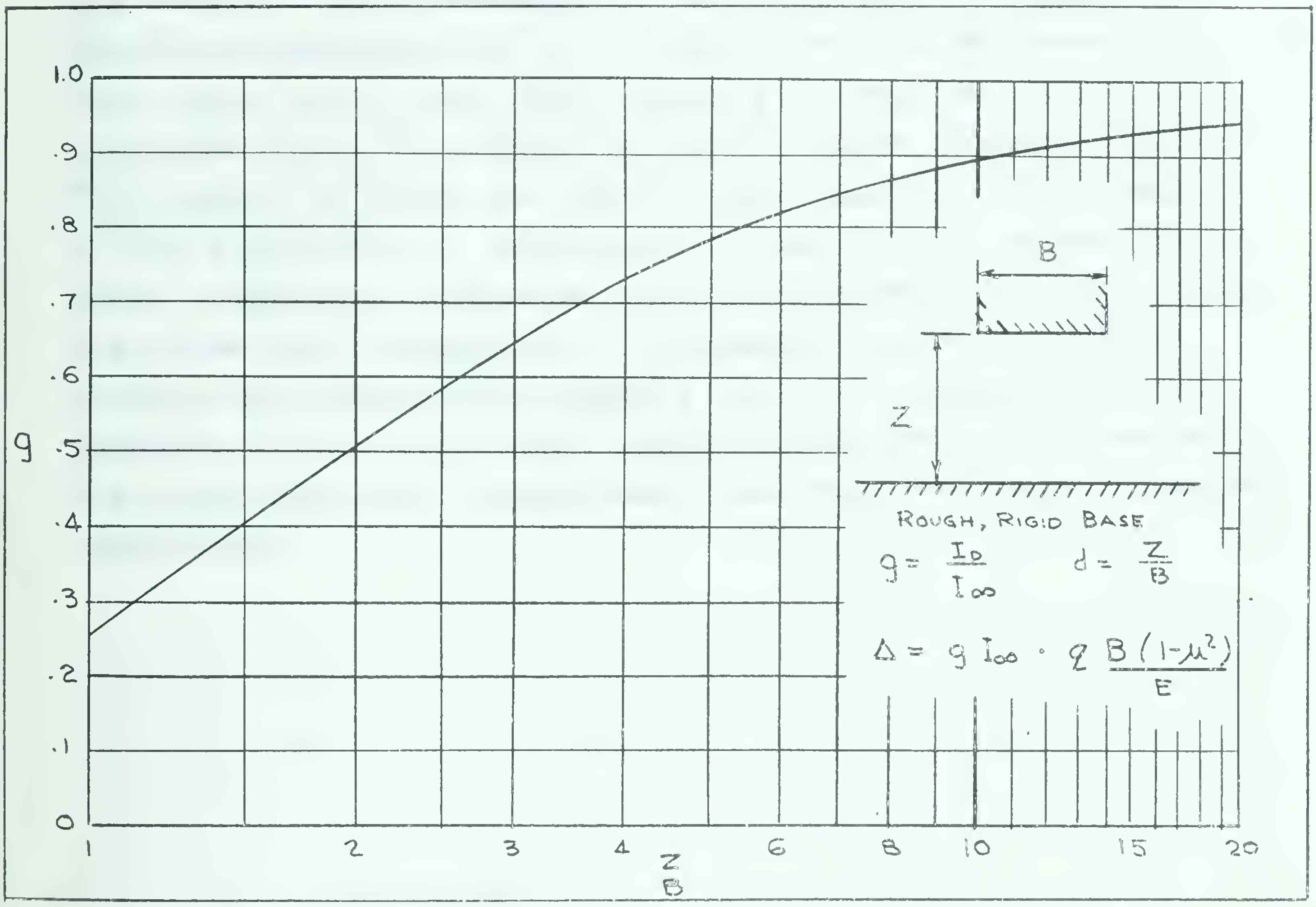


Figure 3-11 Correction Coefficient $g = I_D/I_\infty$

No reference has been found which analyses the effect of the rigid walls of the mould. However several workers have examined the effects of adjacent piles when analysing the behaviour of pile groups. Using the concept of a "mirror" image of stress from an imaginary pile at the same distance from the mould wall as the real model pile (similar to the approach for boundary conditions in well drawdown problems eg. Todd, 1959, pg 102) the effect of the proximity of the wall of the mould can be compared to

the effects of pile spacing in groups. The imaginary pile spacing is equal to twice the distance of the pile to the wall. For the mould used in the model pile testing program the ratio of this distance to the pile shaft diameter b, would be approximately 9.4. Model tests by Whitaker in clay soils have shown that beyond pile spacing ratios of approximately 8 the effect of interference is negligible. This cannot be taken as direct proof that the rigid walls of the mould have no effect on the model pile, since the mould completely surrounds the pile whereas in a pile group the piles are arranged in a rectangular pattern. Nevertheless the comparison suggests that the influence of the boundary on the pile shaft capacity may not be too serious. The only real proof would come from tests in larger diameter containers.

CHAPTER IV

MODEL PILE TESTING PROGRAM

4.1 Scope

The model pile testing program described herein was developed to measure the following variables during the loading of a pile:

1. The total load and penetration of pile top.
2. The distribution of load along the pile shaft during loading.
3. The load carried by the base of the pile.
4. The shear strength and stress-strain properties of the soil along the shaft and below the pile base.

The purpose of this chapter is to describe the design and construction of the model pile and other special equipment and to outline the testing procedures used for the four model pile tests.

4.2 General Considerations

The model pile, soil and container were designed with the following ideal requirements in mind, so that:

1. The pile would represent a field pile in deformation characteristics, that is have approximately the same ratio of stiffness of model pile to soil as a typical field pile.
2. A variety of base diameters could be used.
3. Loading could be both by maintained load in increments or at a constant rate of penetration.

4. Strains could be measured along the shaft.
5. The base load could be measured.
6. There would be no influence of the container on pile behaviour.
7. The soil would be purely cohesive.
8. Reasonable quantities of soil would be required for each test.

The decision to use a loading device which could apply load in increments or at a constant rate of strain immediately restricted the general dimensions of the pile container since the most suitable machine available was a Farnell loading press. This allowed a maximum diameter of 10 or 11 inches and height of about 16 inches for the soil container or mould.

The practical problems associated with construction of a shaft which could be split to permit installation of the necessary strain gauges and which would develop a substantial shaft resistance lead to the choice of a one-inch diameter brass tube with a length of about 15 inches. Brass was chosen because of its relatively low modulus of elasticity (15×10^6 psi), adequate surface hardness for protection against damage, good machinability and high resistance to corrosion. The wall thickness (.05 inches) was selected to give adequate strain response for the loads expected, sufficient thickness for construction purposes and a reasonable approximation to the behaviour of a prototype pile.

The construction of the base of the pile provided for a cell to measure base loads and for interchangeable bases. The load cell was made as short as possible in order to transfer the pile load from shaft to base at a point low

enough to avoid changing the strain distribution along the shaft. The design had also to provide sufficient area for strain gauges, be sensitive to small loads and have essentially the same stress-strain response as the pile shaft, thus not creating an unnatural discontinuity at the junction of the shaft to the base.

Two base diameters were chosen; 1 inch and 2 inch. The two inch diameter base was judged to be the largest that could be used without encountering serious interference from the rigid base of the mould for practical depths of the soil below the pile base, and was flared to represent a pile bell*.

The highly plastic, impermeable Lake Edmonton clay was chosen for the test soil on the assumption that for the relatively high strain rates to be used it would behave essentially as a purely cohesive soil. The soil properties are given in Appendix A. The dimensions of the mould required approximately 80 pounds of air-dry soil for each test.

4.3 Construction of Model Pile

The model pile as constructed is shown in Figure 4-1 and 4-2; a cross-section of the pile and some technical construction details are given in Appendix B. The pile consists of a loading cap, a split shaft of brass tubing one inch in diameter and walls 0.0526 inches thick, a load cell and two bases, one inch and two inches in diameter.

The load cell design selected was a solid brass cylinder 1/2 inch diameter and 1/2 inch high, threaded at both ends. The completed load cell is shown in detail in Figures 4-3 and 4-4 and on drawing M-1a in Appendix B.

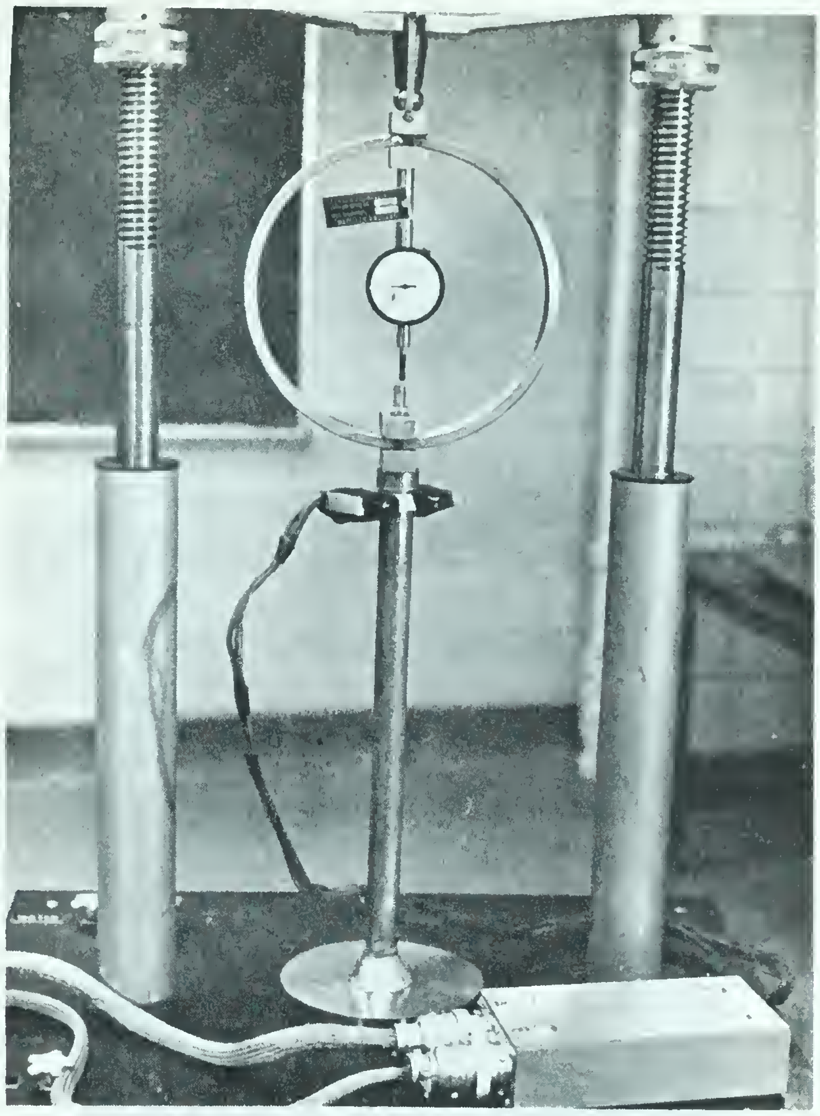


FIGURE 4-1 Model Pile

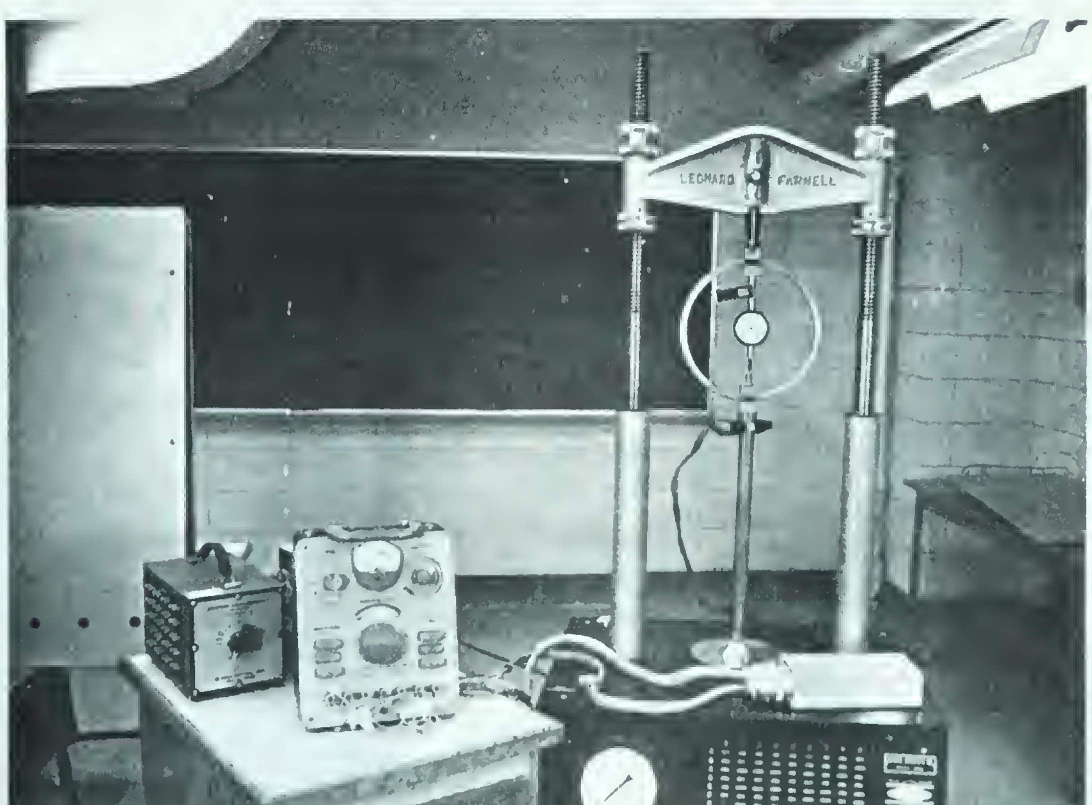


FIGURE 4-2 Model pile calibration test in Farnell loading press. Baldwin strain indicator connected through 20-point switch box to pile gauges.

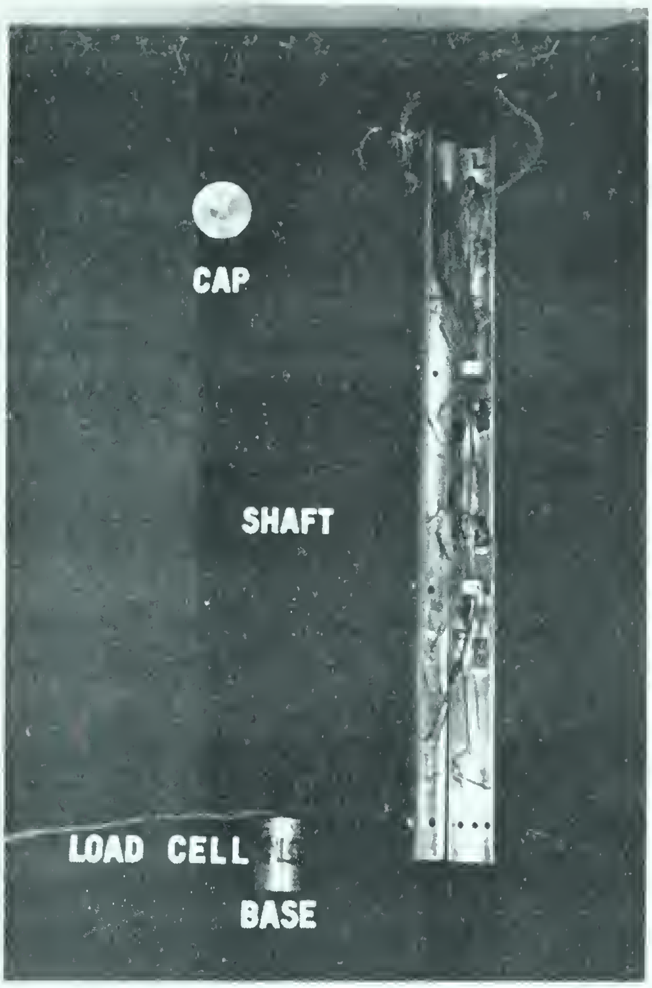


FIGURE 4-3 Interior details, cap, load cell and 1 in. base of model pile.

FIGURE 4-4 Load Cell, 1 in. base and dummy gauge.

FIGURE 4-5 2 in. base, foil shield and rubber membrane.

The upper end of the load cell proper screwed into a brass ring which in turn was held inside the shaft by 8 steel screws. The two bases screwed onto the lower end of the load cell as shown in Figures 4-4 and 4-5.

The two machined halves of the 15 inch long shaft are held together by the threaded loading cap, four screws into each of the two circular spacer rings at the third points along the shaft and by the load cell ring at the base.

4.4 Strain Gauge Instrumentation

Electrical resistance strain gauges, gauge length 1/8 inch Budd Metafilm type C 12-121, were placed at 6 levels on the pile at approximately 2 1/2 inches on centres. At each level a total of 4 gauges were placed; 2 vertical and 2 horizontal on the inside of the tubing.

The 1/2 inch diameter brass load cell had 4 of the same strain gauges mounted vertically 90° apart around the periphery. Mounted above the load cell ring but free from the shaft wall was a small brass ring upon which was mounted a single strain gauge to act as a dummy for the resistance circuit.

Each of the 28 active strain gauges had one individual and one common wire. The dummy gauge had two independent wires. The wires were brought out through 2 holes near the loading cap and were supported by an aluminium clamp. A view of the partly opened shaft showing details of construction and strain gauge installation is given in Figure 4-3. All strain gauges were given 3 coats of Budd GW-1 Waterproofing compound before using.

The strain gauge wires were connected to two-20 point switch boxes and through these to a Baldwin SR-4 Type N Strain Recorder. Because the model pile was frequently disconnected from the strain measuring apparatus, an intermediate connection using 2 male and female Cannon threaded connectors was used. The general layout of the pile, connectors, strain recorder and one of the two switch boxes is shown in Figure 4-2.

The load cell and shaft both had strain responses of about 0.9 micro-inches/inch/kilogram indicating that there would be no strain discontinuity at the base of the pile.

4.5 Mould

For a preliminary test a mould was fabricated from a 16 inch length of 10 inch nominal diameter steel pipe by welding a flat base on one end. To enable the soil to be removed from around the pile after the test without disturbing the failure surfaces a second split-section mould was constructed of 3/16 inch thick aluminium to the dimensions shown in Appendix C and as illustrated by Figure 4-6. The light weight of this mould made handling much easier.

4.6 Pile Calibration

The strain gauges of the load cell and the assembled model pile were calibrated against proving ring loads; Figure 4-2; page 67 shows the set-up for calibration of the pile. The proving ring itself had been calibrated by a dead-weight method. During calibration of the pile, the vertical gauge R6 (R for right half of shaft, 6 for sixth vertical gauge from top of pile) behaved erratically and so no calibration curve was drawn. This was not considered a serious loss since the connection between the

load cell and shaft was only slightly below this gauge and the load cell would thus give the shaft load at that point reasonably well.

Just after waterproofing, the horizontal gauges R7 (7 for 1st horizontal strain gauge on the right half of the pile shaft) and L 10 (3rd horizontal strain from top on the left half) did not operate properly. It was felt that their loss was not serious enough to warrant opening up the shaft section and risking further damage to other strain gauges and so these two gauges were inoperative throughout the test program.

4.7 Preliminary Soil Tests

Preliminary static compaction tests in a Minature Harvard Compaction mould (1 5/16 inch diameter by 2.816 inches long) were made on the Lake Edmonton clay at various moisture contents and densities for various undrained shear strengths. The samples were compacted in one layer by gradually increasing the load on a piston of the diameter of the mould to give the desired unit pressure and holding this pressure constant for one minute. The samples were measured and weighed and then subjected to unconfined compression tests.

4.8 Soil Preparation

For each test a shear strength for the shaft soil and base soil were chosen and from the preliminary soil tests a corresponding water content and density. The soil was placed in the mould in lifts of about 4 inches so corresponding amounts of soil were prepared at the chosen moisture content.

The soil was initially air-dried and then ground in a rotating face-to-face crusher to minus a No. 40 U.S. Standard sieve. Using a pre-weighed amount of the ground air-dry soil, correcting for hygroscopic moisture, water was added by a spray bar as a fine mist and mixed into the soil. The final moisture content was reached when the soil and water in the mixing pan was at the exact weight required as determined by pre-calculation. This method produced a uniform distribution of moisture very close to the desired water content. The prepared soil was stored in polythene bags in a humid room to further ensure uniformity of moisture.

4.9 Installation of Pile and Soil in Mould

To achieve the necessary soil strength uniformly over the mould cross-section, it was felt that a static compaction method would produce the best result. For the soil below the pile base this method was achieved easily by the use of a circular piston. Around the pile shaft however, it was necessary to use a circular split plate, 1/2 inch thick, which fitted around the pile and was loaded by a steel cylinder large enough to completely surround the pile. The strain gauge wires were led out through a slotted cap at the top of the loading cylinder. Loading was done by a Tinius Olsen hydraulic testing machine.

The installation procedure consisted of weighing out the exact amount of prepared soil to give the design density for the lift thickness required, placing this amount in the mould and loading the piston to give the net thickness. The load was maintained on the piston for at least one minute. The soil below the base was placed in one lift; the soil around the shaft in three-

4 inch lifts. Placing the soil for the last lift at the top of the mould required several preliminary compactions in order to get all the soil in place. This was because of a limited space between the top of the mould and the clamp on the pile.

The surface of the soil in the mould was covered by two thicknesses (a total of about 1/64 inches) of a synthetic rubber coating, Duncote No. 101 HP, manufactured by Dunlop of Canada Ltd., and the mould and pile stored in the humid moist room (relative humidity approximately 70%) where it was anticipated that shear stresses induced in the soil by the compaction processes would adjust to an equilibrium condition. This was checked by readings on the pile strain gauges which in a few days reached a constant value.

The pile was installed with its base on the first layer centred in the mould and held vertically until sufficient soil had been placed by hand to hold it. Soil was prevented from entering the gap between shaft and base by a thin rubber membrane cemented in place. For the flared 2 inch diameter base, a shield of aluminium foil was placed over the upper surface to prevent soil adhesion; the rubber membrane and aluminium shield may be seen in Figure 4-5.

A total of four tests were carried out, numbered A-1 to A-4. Tests A-1 and A-4 used soil of approximately the same undrained shear strength for both the shaft and base soils. Test A-2 and A-3 used a soil of greater strength below the base than around the shaft. The one inch diameter base was used for tests A-2 and A-4 and the 2 inch flared base was used for tests A-1 and A-3.

4.10 Pile Testing

When the check of all strain gauges showed that the readings were essentially constant the mould and pile were brought out of the moist room and set up in the Farnell loading frame as shown in Figures 4-6 and 4-7.

Load was transferred to the pile from the loading head by the proving ring and a ball joint. Penetration of the pile top was measured by a dial gauge at the bottom of the proving ring. The dial gauge was accurate to 0.0001 inch with a 1 inch travel and rested on a support on the top of the mould.

The loading procedure consisted of increasing the strain on the pile at a rate of 0.0061 inches per minute up to a given load; maintaining this load constant by continued penetration by hand while the various strain gauges were read and then continuing at the constant rate of strain to a new load. Time readings from the start of loading were taken at the beginning and end of each set of strain gauge readings.

The strain gauge readings took about 7 minutes for each set of 28 gauges. At low loads both the load and penetration remained unchange during readings. However as the loads increased it became necessary to increase the penetration to maintain the load while the readings were taken. Eventually continuous penetration was necessary to maintain the load. The loading rate was kept constant at 0.006 inches per minute until a maximum load was reached at which time all strain gauges were read.

The pile was then unloaded by two or three stages, to zero load, with strain gauge readings at each pause.

Readings were also taken at intervals afterward until they showed no further change, usually in a matter of a day or so.

For the first two tests, A-1 and A-2, the pile was reloaded and unloaded a second time before dismantling; the second two tests, A-3 and A-4, were dismantled immediately after the first loading and unloading cycle.

4.11 Soil Sampling

In order to sample the soil for strength and density determinations, four sampling tubes were constructed of 1 5/8 inch, thin-wall Shelby tubing. Details of the sampler are shown in Appendix D.

The sample tubes were forced to the full depth of the soil using a Baldwin 100 ton loading press. Two tubes were located as close to the pile as possible. The other two tubes were located half way between the mould and pile. A wax plug was poured on the top of the soil within the tube as soon as sampling was completed. The pile, mould and sampling tubes still in place were taken to the moist room for dismantling.

4.12 Dismantling Mould

One half of the mould was detached and the soil split and removed to show the soil-pile interface and the form of the failure surface below the base. Moisture contents were taken from the surface of the soil just below the rubber paint layer to determine whether or not drying out had occurred. The results showed that very little moisture had been lost from the top of the soil.

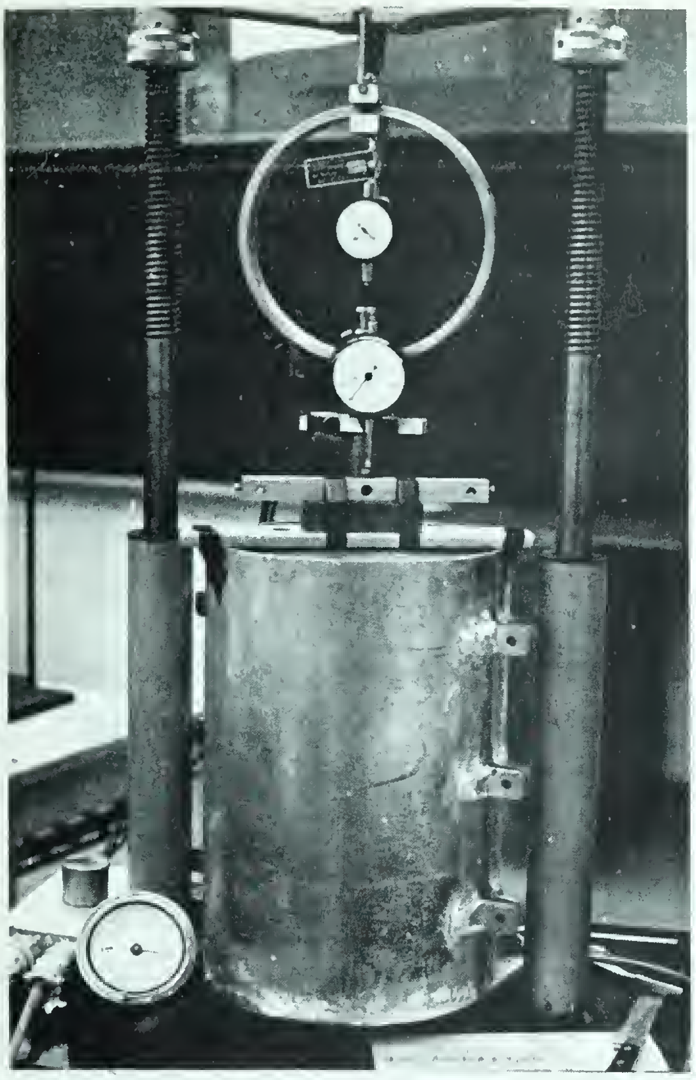


FIGURE 4-6 Model pile in mould for test.

FIGURE 4-7 General layout of model pile test.

After examination of the pile and soil surfaces all soil was removed from the mould and the lower ends of the sample tubes sealed with wax.

4.13 Soil Strength Tests

The stress-strain behaviour of the soil was determined by unconfined compression tests. Three or four samples of the soil below the base and 8 or 9 samples of the shaft soil were tested for each loading test.

The tests were carried out using triaxial compression apparatus developed by Geonor. Each sample was sheathed in a rubber membrane and tested within a plexiglass cell to reduce evaporation; there was no confining pressure other than atmospheric.

The loading rate of strain of the Geonor press crosshead was 0.0160 inches per minute; this gave an actual strain rate of about 0.4% per minute for the specimens used (3.9 cm. diameter by 8.0 cm. long).

In Figures 4-8 and 4-9 are shown views of the data form used and a typical failure pattern for sample A-3-3-4.

To substantiate the assumption of constant shear strength in undrained conditions regardless of the magnitude of the principal stresses (the $\varphi=0$ condition) a single confined triaxial test at a cell pressure of 9.0 kg/cm^2 was carried out for test A-3 and four confined triaxial tests at cell pressures of 1.0, 3.0, 5.0 and 7.0 kg/cm^2, for test A-4. The tests were carried out at a strain rate of approximately 0.4% per minute.

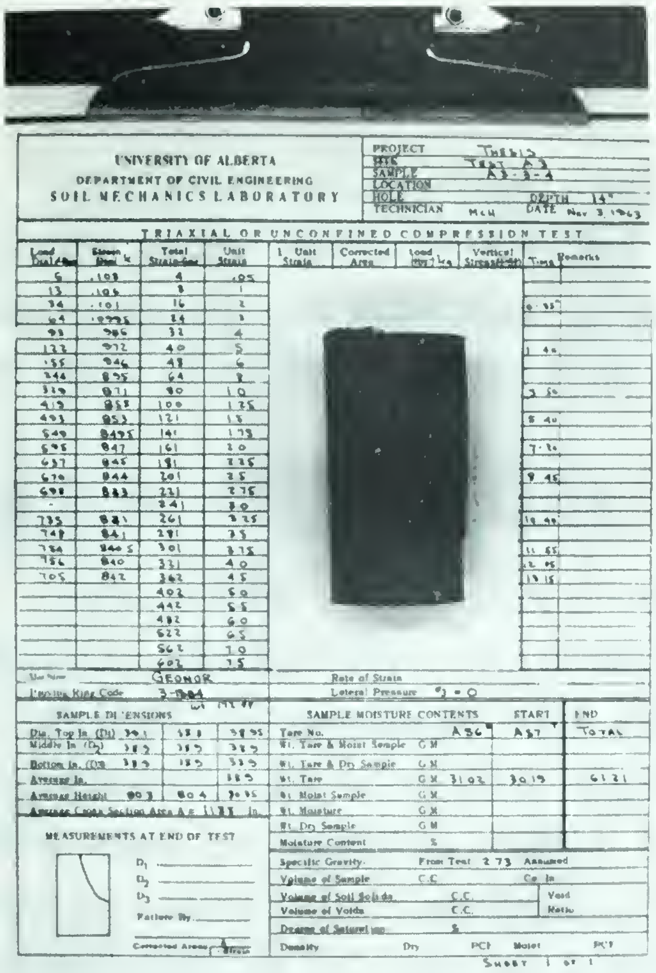


FIGURE 4-8 Unconfined compression test data and sample failure.

FIGURE 4-9 Close-up of failure surface showing slicken sides.

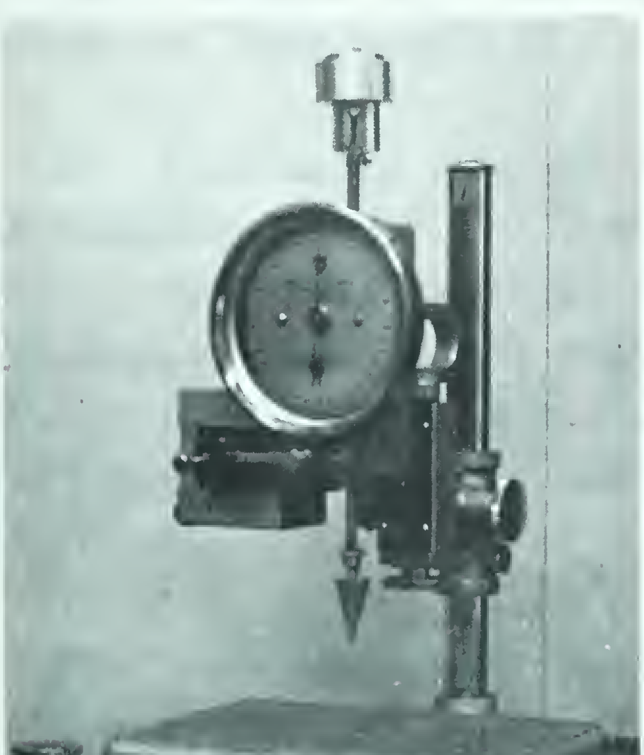


FIGURE 4-10 Cone Penetrometer, cone angle 30°

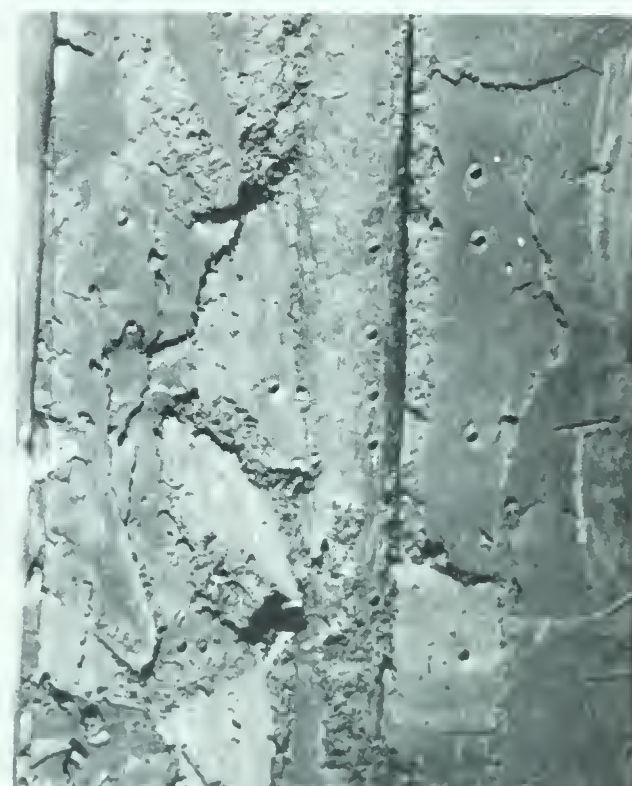


FIGURE 4-11 Cone penetration in soil around shaft. Test A-4.

The samples for all tests were measured and weighed and the water content of the entire sample taken. This data enabled calculation of void ratio, water content, dry and bulk density and percent saturation.

4.14 Cone Penetration Tests

After the first three tests had shown very low skin friction between the pile shaft and soil it was decided to attempt to evaluate the shear strength of the soil in the vicinity of the pile surface. The unconfined compression tests were on samples taken at least one and one half inches from the shaft surface and therefore were possibly not representative of the soil strength at the pile surface.

Accordingly, the cone penetrometer shown in Figure 4-10 was used to determine relative undrained shear strengths at the soil-pile interface, in the centre of the space between pile and mould wall, and of the unconfined compression test samples.

The cone angle was 30° and the total weight 400 grams, thus comparing with one of the standards used by Hansbo (1957). The holding device was actually designed for the standard penetration test in asphalt analysis, but was easily modified for the cone penetration test by interchanging the points and adding weight to the penetrometer shaft.

The test was run by placing the point of the cone at the soil surface using an adjusting screw, and allowing the cone to drop freely into the soil. The penetration of the cone was measured directly by the dial in millimeters. Figure 4-11 shows the cone indentations for soil from around the shaft of Test A-4.

4.15 Skin Friction Tests

To further evaluate the skin friction between the soil and the pile shaft in Test A-4, two plates of the same type of brass and with the same surface texture as the model pile were pressed onto the top of the soil when the upper lift of soil was placed. The intention was to test these in a direct shear test by forcing a shear plane at the soil-plate interface, using the standard direct shear apparatus. The soil sample was placed in the upper half of the shear box.

In attempting to trim the samples from the mould only one remained unbroken and the plate slipped on the soil surface before the test was started. There was thus no intimate bond created between the metal and the soil. The sample was however tested under a normal load of 1.0 kg/cm^2.

Additional series of tests were made to evaluate skin friction using soil left over from test A-4. The same two brass plates were pressed onto each face of the soil as compacted in a 6 inch diameter mould. The soil was compacted to the same density and at the same unit pressure as the soil in the pile test. Careful trimming yielded two specimens which were tested at normal loads of 1.0 and 2.0 kg/cm^2 respectively. In the case of the test at 1.0 kg/cm^2, a re-run was made by simply sliding the plate back to its initial position.

As well as these two tests, one additional test was made by "consolidating" a trimmed sample of the soil onto one of the plates right in the shear box, under a load of 2.0 kg/cm^2, and one test made of the soil itself

in direct shear under the same load. The consolidation test consisted of measuring vertical strain readings at given time intervals after the vertical load was placed until no measureable movement occurred. The data obtained from the direct shear tests are included in Appendix I.

4.16 Pile Materials Testing

In order to determine stresses from the strain gauges it was necessary to evaluate the stress strain properties of the metal used in the shaft and load cell. The modulus of elasticity and Poisson's ratio of the pile shaft material were determined by compression tests on a short piece of the original brass tubing 1 1/2 inches long. Two vertical and two horizontal strain gauges were placed diametrically opposite each other around the centre-line. The strain gauges were Budd Metafilm type C 12-141B, 1/4 inch gauge length, 120 ohms, gauge factor 2.08. A dummy gauge was placed on a similar piece of brass. The cylinder was loaded in the Farnell loading press using the same proving ring as for the pile tests. Strain readings were taken by the Baldwin SR-4 Type N Strain Recorder.

The modulus of elasticity of the load cell material was determined from a solid cylinder 1 inch diameter by 3 inches long with three vertical SR-4 strain gauges mounted 60° apart on the centre-line. The gauges were SR-4 type A-5-S6 with a one-half inch gauge length and a gauge factor of 2.00. A dummy gauge was placed on a separate piece of similar brass. The cylinder was loaded in a Baldwin Universal 10 ton loading press with the strain read by the Baldwin SR-4 Type N Strain Recorder.

The results of both tests on the brass tube and cylinder are shown in Appendix E.

4.17 Model Pile Behaviour

The analysis of the model pile tests consists of interpreting the readings from the strain gauges on the shaft and load cell.

For the load cell, see Figure 4-4, the basic equation for the base load Q at a given average unit strain is:

$$Q = \epsilon A_c E_c \qquad 4\text{-}1$$

where A_c is the cross section area of the cell and E_c is the modulus of elasticity of the metal (brass). The average unit strain is obtained by averaging the readings of the four gauges on the shaft. Trial calculations for load did not compare well with the observed loads even when using a measured value of the modulus of elasticity of the brass. The discrepancy may be due to uneven distribution of stress over the load cell cross section when screwed into the load cell ring. The load cell was therefore calibrated against known loads after installation in the pile. Reported base loads have therefore been found from a calibration curve.

The stress-strain behaviour of the shaft in the soil is not as simple as that of the load cell since the outer boundary of the shaft is not a free surface during the test. The vertical load on the pile at any cross section cannot be calculated from the unit strain of a vertical strain gauge alone but must be calculated using the basic equations of elastic deformation.

For a thin-walled cylinder of modulus of elasticity E_p and Poisson's ratio μ with vertical axis in the h direction, the vertical and horizontal stresses in the wall, σ_h and σ_θ respectively, are given by

$$\sigma_h = \frac{E_p}{(1-\mu^2)} (\epsilon_h + \mu\epsilon_\theta) \qquad 4\text{-}2a$$

and

$$\sigma_\theta = \frac{E_p}{(1-\mu^2)} (\epsilon_\theta + \mu\epsilon_h) \qquad 4\text{-}2b$$

where ϵ_h and ϵ_θ are unit strains in the vertical and circumferential directions respectively (neglecting any change in the thickness of the wall itself) (Stippes, 1961, pg 265). The vertical force or load at depth h is

$$Q_h = \sigma_h A_p \qquad 4\text{-}3$$

where A_p is the area of the cylinder or model pile. From a consideration of statics of a longitudinal section along the pile it can be shown that the uniform pressure of the soil normal to the cylinder wall is given by

$$P_h = \frac{2t}{b} \sigma_\theta \qquad 4\text{-}4$$

where t is the wall thickness and b the diameter of the pile shaft.

CHAPTER V

TEST RESULTS AND DISCUSSION

5.1 Scope

The purpose of this chapter is to present the results of the model pile testing program and to analyse them in terms of pile behaviour. The results will be discussed qualitatively first and then analysed quantitatively in detail.

Where possible the results are plotted graphically in the form of dimensionless values so that they can be easily compared.

The four tests, A-1 to A-4, are renumbered as follows for identification purposes in the computer analysis:

A-1 a	111
A-1 b	112
A-2 a	121
A-2 b	122
A-3	130
A-4	140

5.2 Qualitative Aspects of Tests

After each of the four tests the soil was carefully removed from around the pile in two halves to reveal the nature of the failure surfaces of the shaft and the base soils. Figures 5-1 to 5-4 show the results of test A-3, which is typical of all the tests.

Figure 5-1 shows that some soil adhers to the pile but that about 70% of the shaft is bare indicating that failure in skin friction took place mainly between the pile surface and the soil. The adhesion of the soil is greater near the top of the shaft and this is consistent with the distribution of negative stresses shown in Figures 3-9b and 3-10b if it can be assumed that the lateral stresses are proportional to the vertical stresses predicted by the Mindlin-Ruderman analysis. The soil surface can be seen in Figures 5-2 to 5-4; Figure 5-4 shows some slickenside* surfaces (dull in the photograph) similar to those in the unconfined compression test of Figure 4-9. The direct shear tests on the soil in Test A-4 showed similar failure surfaces. The failure surface of the direct shear test No. F 2-2-B1 in which the soil was compressed onto the brass plate under a load of 2 kg/cm^2, also showed noticeable slickensides; the other direct shear tests had almost clean failure surfaces.

The nature of the failure surface of the soil below the base was similar for all tests and is illustrated in Figures 5-2 and 5-3. The base has penetrated vertically into the soil causing a well defined plug of soil to form. The plug is more or less circular at all horizontal cross sections with a smooth shiny surface. The vertical cross-section is illustrated in Figure 5-5. The lower boundary of the plug becomes indistinct at a depth about equal to one half the base diameter but does indicate a general downward and outward direction of the failure surface. The angle between the plane of the base and the failure surface at the edge of the base is about 75^o for all tests. The failure pattern does not fit the shape of the zones proposed by Meyerhof for deep foundations

FIGURE 5-1 Soil adhering to pile surface, Test A-3.

FIGURE 5-2 Soil failure surface after pile removed, Test A-3.

FIGURE 5-3 Close-up of base penetration (.52in.) Test A-3

FIGURE 5-4 Close-up of shaft failure surface, Test A-3.

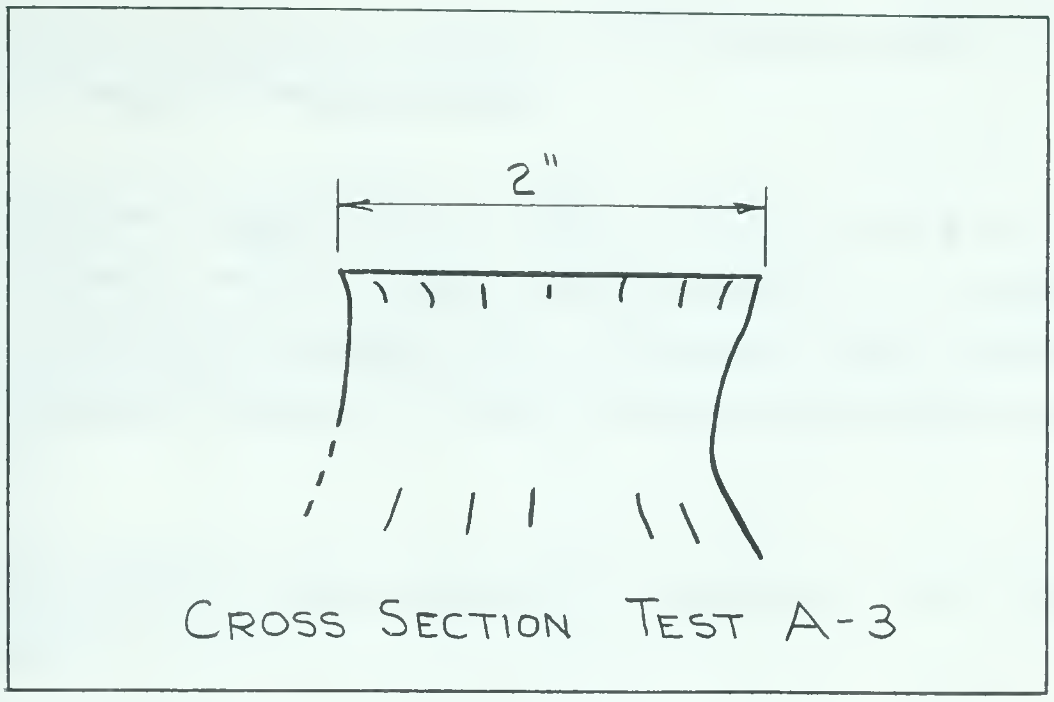


Figure 5-5 Shape of Base Failure Surface

in a purely cohesive soil, as shown in Figure 3-4, page 43, nor of the classical models given by Prandtl and Terzaghi. The explanation for this may lie in the influence of the boundaries of the mould, or in the fact that although the analyses of Prandtl, Terzaghi and Meyerhof are statically correct and give reasonably accurate bearing capacities, they are not rigorously correct for the velocity field conditions which apply at plastic equilibrium (Shield, 1955, pg 268; Cox, 1961, pg 3). In other words the plastic zones were drawn without proper reference to the stress-strain properties of the soil. The failure surface shape is similar to those which result from deep penetration of dies into a soft metal such as lead (Kudo, 1961).

5.3 Data

For reasons of space the original data sheets for the pile gauge strain readings for the calibration and pile tests and the soil tests will be bound as a separate volume. A computer print-out of gauge strain readings for

both calibration and pile tests is given for each in Appendices F and G respectively.

Two typical compression tests for the soils of each of the pile tests are included in Appendix H. A print-out of the computer program for a best-fit Mohr envelope for the undrained triaxial tests is also included in Appendix H.

All the direct shear tests for Test A-4 are included in Appendix I.

5.4 Soil Strength Test Results

A typical data page for the unconfined compression tests is included here as Figure 5-6 (Sample A-3-3-4). This is the same data sheet shown in Figure 4-8, page 78. The sheet shows the load and strain dial readings, the corrected area $A_c = A_o/(1- \epsilon)$, the compressive stress, σ_d and the stress ratio at failure τ/τ_f. Also shown is the data for moisture content, density, void ratio and percent saturation. Calculations for these are as given in standard textbooks on soil mechanics. Six typical data sheets, two for each pile test, are included in Appendix I.

Table 5-1 shows the summaries of all the unconfined compression tests for each of the four pile tests. The average coefficient of variation* for the unconfined compression tests for the shaft soil is $\pm 4.0\%$ ($\pm 3.74\%$ to $\pm 4.02\%$), and for the base soils is $\pm 5.0\%$ ($\pm 2.63\%$ to $\pm 8.60\%$). Summary sheets for the individual compression tests for all pile tests are given in Appendix I.

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A3
SAMPLE	A3-3-4
LOCATION	
HOLE	DEPTH 14"
TECHNICIAN MCH	DATE Nov. 3, 1963

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d /~~Rdg~~	~~Strain Dial~~ k	Total Strain ~~(in.)~~	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kg	σ_d Vertical Stress ~~(PSI)~~	TIME	Remarks	$\frac{\tau}{\tau_f}$
6	.108	4	.05		11.88	0.64	.05			.01
3	.106	8	.1		11.89	1.38	.12			.02
34	.101	16	.2		11.90	3.40	.29	0-35"		.06
64	.0995	24	.3		11.92	6.39	.53			.10
93	985	32	.4		11.93	9.16	.77			.15
22	972	40	.5		11.94	11.858	.99	1 40		.19
55	946	48	.6		11.95	14.66	1.23			.24
44	895	64	.8		11.98	21.84	1.82			.354
29	871	80	1.0		12.00	28.66	2.39	3 50		.466
19	858	100	1.25		12.03	35.95	2.99			.582
93	853	121	1.5		12.06	42.05	3.49	5-40		.680
49	8495	141	1.75		12.09	46.64	3.86			.752
95	847	161	2.0		12.12	50.40	4.16	7-20		.810
37	845	181	2.25		12.15	53.83	4.43			.864
70	844	201	2.5		12.19	56.55	4.64	8 45		.903
98	843	221	2.75		12.21	58.84	4.82			.938
-		241	3.0		12.25	-	-			
35	841	261	3.25		12.28	61.81	5.03	10 40		.981
48	841	281	3.5		12.31	62.90	5.11			.995
54	840.5	301	3.75		12.34	63.34	5.132	11 55	FAILURE	1.000
56	840	321	4.0		12.38	63.50	5.129	12 05		.998
05	842	362	4.5		12.44	59.36	4.77	13 15		.929
		402	5.0							
		442	5.5							
		482	6.0							
		522	6.5							
		562	7.0							
		602	7.5							

hine GEONOR	Rate of Strain $\dot{\varepsilon}_1$ = .315 %/min = 18.9 %/hr
ving Ring Code 3-1584	Lateral Pressure $\sigma_3 = 0$

SAMPLE DIMENSIONS		wt 172.98 / 172.90
Top In. (D1) 39.1	38.8	38.95
dle In. (D2) 38.9	38.9	38.9
tom In. (D3) 38.9	38.9	38.9
rage In.		38.9
rage Height 80.3	80.4	80.35
rage Cross Section Area A = 11.88 In.		

SAMPLE MOISTURE CONTENTS		START		END
Tare No.		AS6 B	AS7 T	TOTAL
Wt. Tare & Moist Sample	G M.	133.94	99.94	233.88
Wt. Tare & Dry Sample	G M.	112.82	85.64	198.46
Wt. Tare	G M.	31.02	30.19	61.21
Wt. Moist Sample	G M.			172.67
Wt. Moisture	G M.	21.12	14.30	35.42
Wt. Dry Sample	G M.	81.80	55.45	137.25
Moisture Content	%	25.82	25.79	25.81

IEASUREMENTS AT END OF TEST

D_1 :

D_2 :

D_3 :

Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 95.46 Cu. In.

Volume of Soil Solids: C.C. 50.27

Volume of Voids: C.C. 45.19

Void Ratio .8989

Degree of Saturation: 78.38 %

Density: Dry: 89.72 PCF. Moist: 112.9 PCF.

SHEET 1 OF 1

Figure 5-6 Unconfined Compression Test DataSheet

TABLE 5-1 SUMMARY OF UNCONFINED COMPRESSION TESTS

Proving Ring No.	Faculty of Graduate Studies	Test Date
Lateral Pressure σ_3 kg/cm^2	MODEL PILE TEST DATA	Base Diameter In.
Avg. Specimen Size cm.	SOIL STRENGTH TEST RESULTS	Net Shaft Length In.

Sample Number		A-1			A 2			A 3			A 4	
Average Depth In.		SHAFT	BASE		SHAFT	BASE		SHAFT	BASE		SHAFT	BASE
σ_d kg/cm^2		2.30	2.42		3.69	4.82		4.00	5.30		2.18	2.12
$c = \tau_f$ kg/cm^2		1.15	1.21		1.84	2.41		2.00	2.65		1.09	1.06
w %		33.46	33.10		29.70	25.85		29.16	25.72		34.38	34.52
e		.996	.983		.882	.897		.877	.890		.991	.984
γ_d lbs/ft^2		85.4	86.0		90.6	89.8		90.8	90.1		85.4	85.9
γ lbs/ft^2		114.0	114.0		117.5	113.1		117.8	113.3		114.8	115.6
S %		91.7	91.9		91.9	78.7		92.6	78.9		94.1	95.8
E kg/cm^2		115	86.1		177	235		218	248		136	104
$\frac{E}{\tau_f} = \frac{E}{c}$		100	64.6		96.1	97.5		109	93.4		125	98.1
ϵ_f %												
$\frac{\epsilon}{1-\epsilon}$ %												
$\dot{\epsilon}_f$ %/min												

LEGEND:

$\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress

$\tau_f = \sigma_d/2 = c$ Shear Strength

w Moisture content

e Void ratio

γ_d Dry unit weight

γ Total unit weight

S Saturation

E Modulus of Elasticity

ϵ_f Unit Strain at failure

$\dot{\epsilon}$ Rate of strain at failure

Remarks:

AVG. E/c = 98.

The confined undrained triaxial test carried out for Test A-3 gave an angle of shearing resistance in terms of total stress of $\varphi=3^{o}$ or in terms of equation 3-1a:

$$\tau_f = 1.80 + \sigma \tan 3^{o}$$

Since this relationship is based on several tests at zero confining pressure but only one test at a confining pressure of 9.0 kg/cm^2, the equation can be assumed as only approximate. For Test A-4 however, the results of four confined undrained compression tests, and the average of 12 unconfined undrained compression tests, are available. The resulting Mohr diagram is shown in Appendix H together with the print-out of a program for the IBM 1620 computer giving the best-fit Mohr failure envelope. The resulting Coulomb equation is:

$$\tau_f = 1.100 + \sigma\,(\tan 1^{o}\ 40')$$

The computer program utilizes the Balmer method for best-fit linear tangent line to the series of Mohr circles, and prints out the tangent of φ, the cohesion intercept c, and then the 95% confinence limits at the mean normal stress, zero normal stress and double the mean normal stress (Haas, 1963, pg C1). The program is stated in IBM Fortran language. The average percent saturations of the A-3 and A-4 soils tested were 93% and 94% respectively.

The results of the two sets of triaxial tests indicates that for all practical purposes the angle of shearing resistance of the soils tested may be taken as zero degrees, even though the soils are not fully saturated. The average percent saturations of all the shaft soils and the base soils in tests A-1 and A-4 are all in the

range of 91.7% to 95.8%. It would therefore be expected that these would behave essentially as if fully saturated. However the base soils of tests A-3 and A-2 have average percent saturations of about 79% and it cannot be implicitly assumed that they would behave as if saturated with a constant shearing resistance at all magnitudes of principal stress.

The one direct shear test performed on the shaft soil of test A-4 (sample F2-1-DS) gave an undrained shear strength of 1.00 kg/cm^2 under a total normal stress of 1.0 kg/cm^2. This is close to the average of 1.09 kg/cm^2 from unconfined compression tests.

Cone penetrometer tests were made on all the unconfined compression tests on A-4 shaft soil. The average unconfined compression strength for the shaft soil was 1.09 kg/cm^2. Using Hansbo's equation (equation 3-2 page 36) the cone penetrometer gave instead an average shear strength of 1.53 kg/cm^2. Assuming that the unconfined compression tests are correct and substituting the actual cone angle and weight, the shear strength of the soil τ_f is given by the empirical relationship:

$$\tau_f = \frac{28.91}{h^2} \qquad 5\text{-}1$$

where h is the penetration of the cone in millimeters.

Using equation 5-1 the undrained shear strengths of the individual penetrometer tests on the shaft soil were calculated and shown at their approximate location on Figure 5-7. The actual imprints of the cone are shown in Figure 4-11.

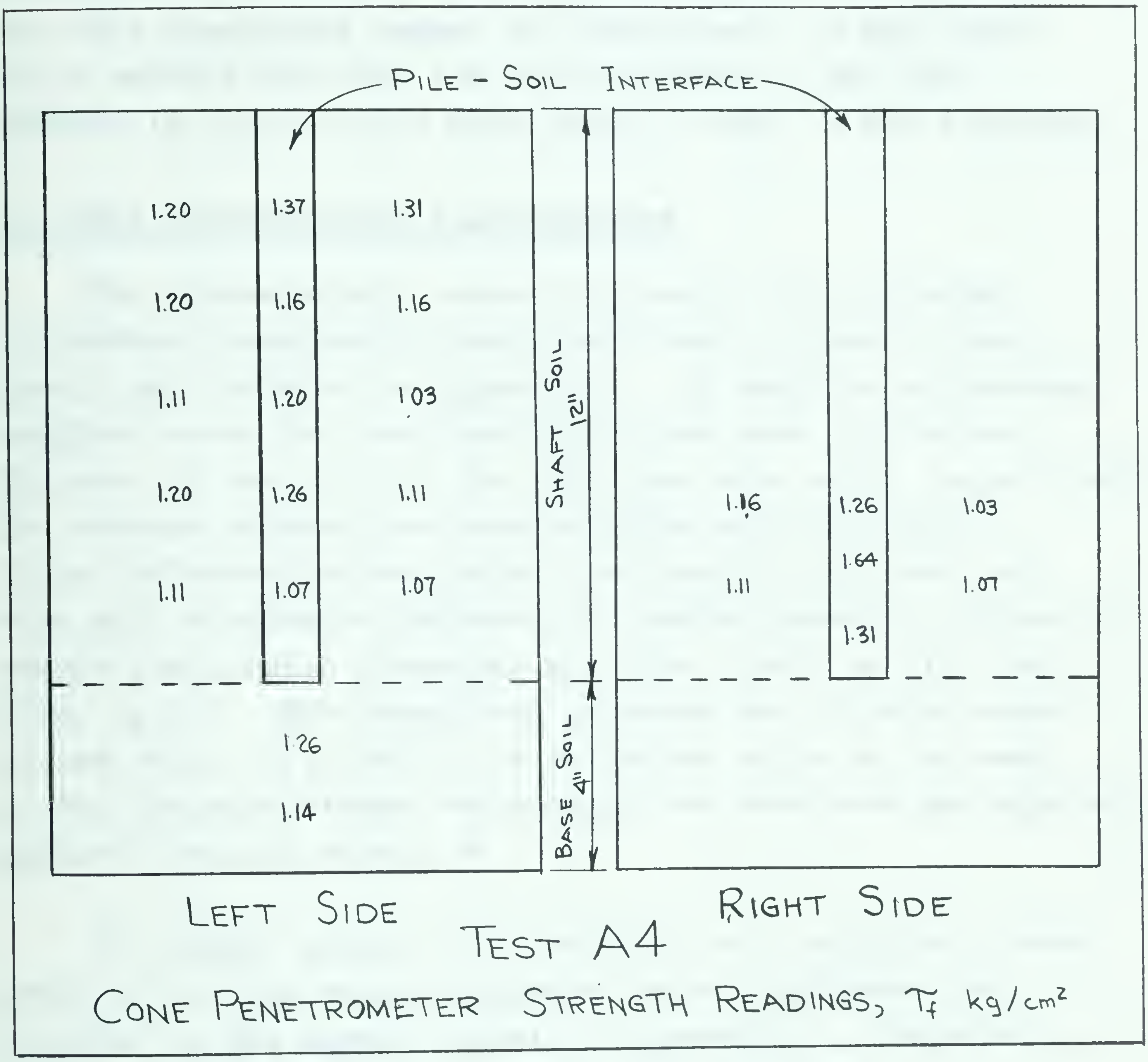


Figure 5-7 Results of Cone Penetrometer Tests

The average shear strength of the soil at the soil-pile interface (1.28 kg/cm^2) is slightly higher than the average shear strength of the soil away from the shaft (1.13 kg/cm^2). A significance test (Moroney, 1951, pg 227) indicates that although the difference between the two strengths is small, approximately 13%, it is statistically highly significant. However whether or

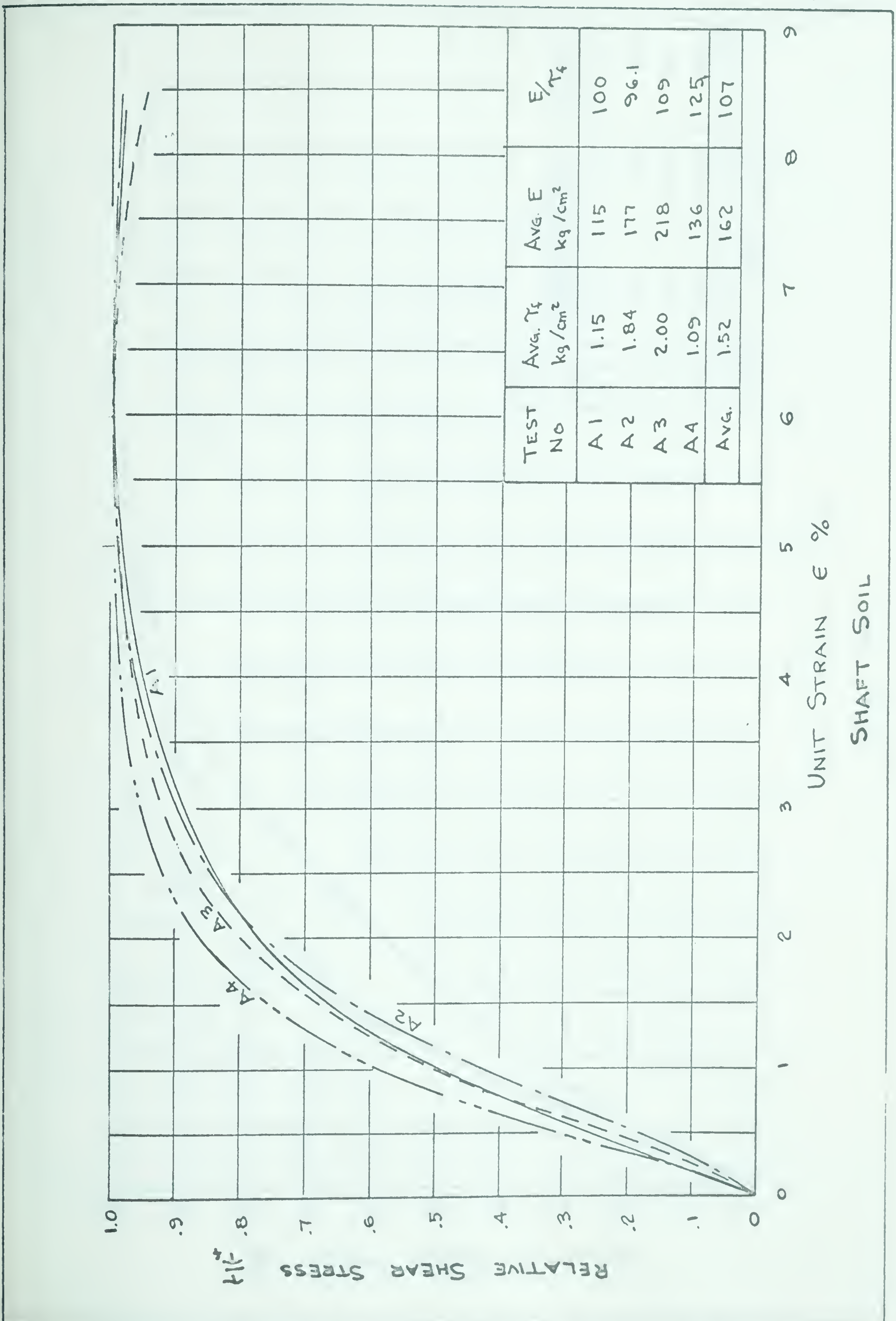
not this reflects a higher soil strength before the test starts or is a result of soil particle reorientation as the test progressed cannot be determined. In any event it is certain that the low skin friction of the pile surface is not due to a weak layer of soil at the interface.

5.5 Soil Stress-Strain Relationships

The stress-strain curves for each of the uniaxial cylindrical compression tests have been plotted by test groups and included in Appendix J. In addition an average weighted curve for each test group has been plotted on Figures 5-8 and 5-9 for the shaft and base soils respectively. The average curves have been weighted by eliminating those individual curves which are irregular or have very high unit strains at failure, the latter probably indicating sample disturbance (Terzaghi and Peck, 1948, pg 110; Ward, 1959, pg 54). For comparison purposes the relative shear stress ratio τ/τ_f (which for saturated soils is the same as the deviator stress ratio σ/σ_d) has been used and plotted against the unit strain ϵ.

The secant modulus of elasticity at a relative stress ratio of 0.5 has been calculated for all specimens and recorded on the summary sheets in Appendix I. Weighted average values are shown on Table 5-1, page 90.

The average secant modulus of elasticity E and the modulus ratio E/τ_f are plotted against average shear strength τ_f in Figure 5-10. The best-fit straight lines through the points show that the secant modulus increases with increasing shear strength and that the modulus ratio is a constant value of approximately 100 for the compacted clays used in the test program. This is the same modulus ratio found by Gibson for tests on a wide range of clays (Gibson, 1950, pg 382).



Test No	Avg. τ_f kg/cm²	Avg. E kg/cm²	E/τ_f
A1	1.15	115	100
A2	1.84	177	96.1
A3	2.00	218	109
A4	1.09	136	125
Avg.	1.52	162	107

Figure 5-8 Summary of Stress-Strain Curves for Shaft Soils

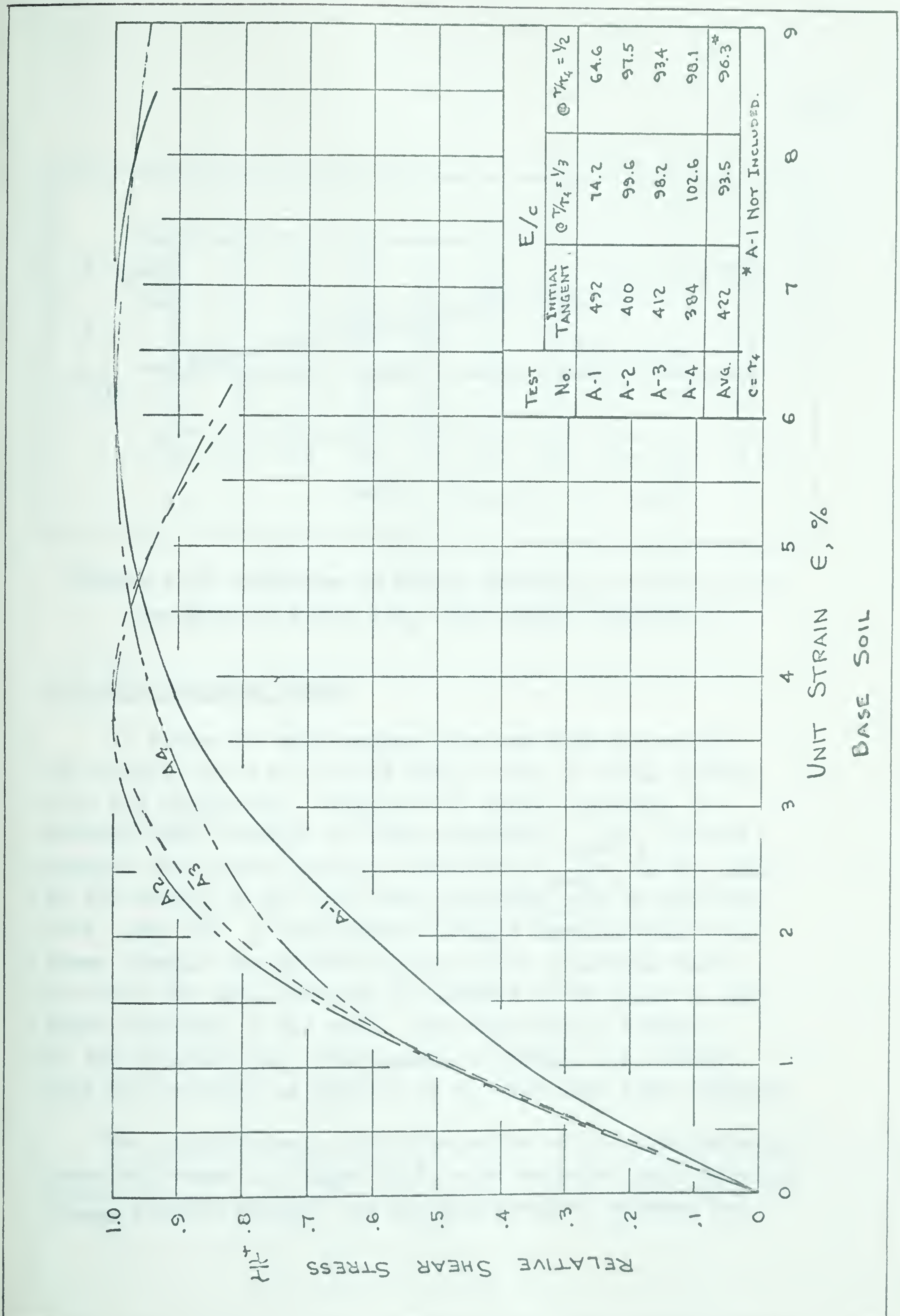


Figure 5-9 Summary of Stress-Strain Curves for Base Soils

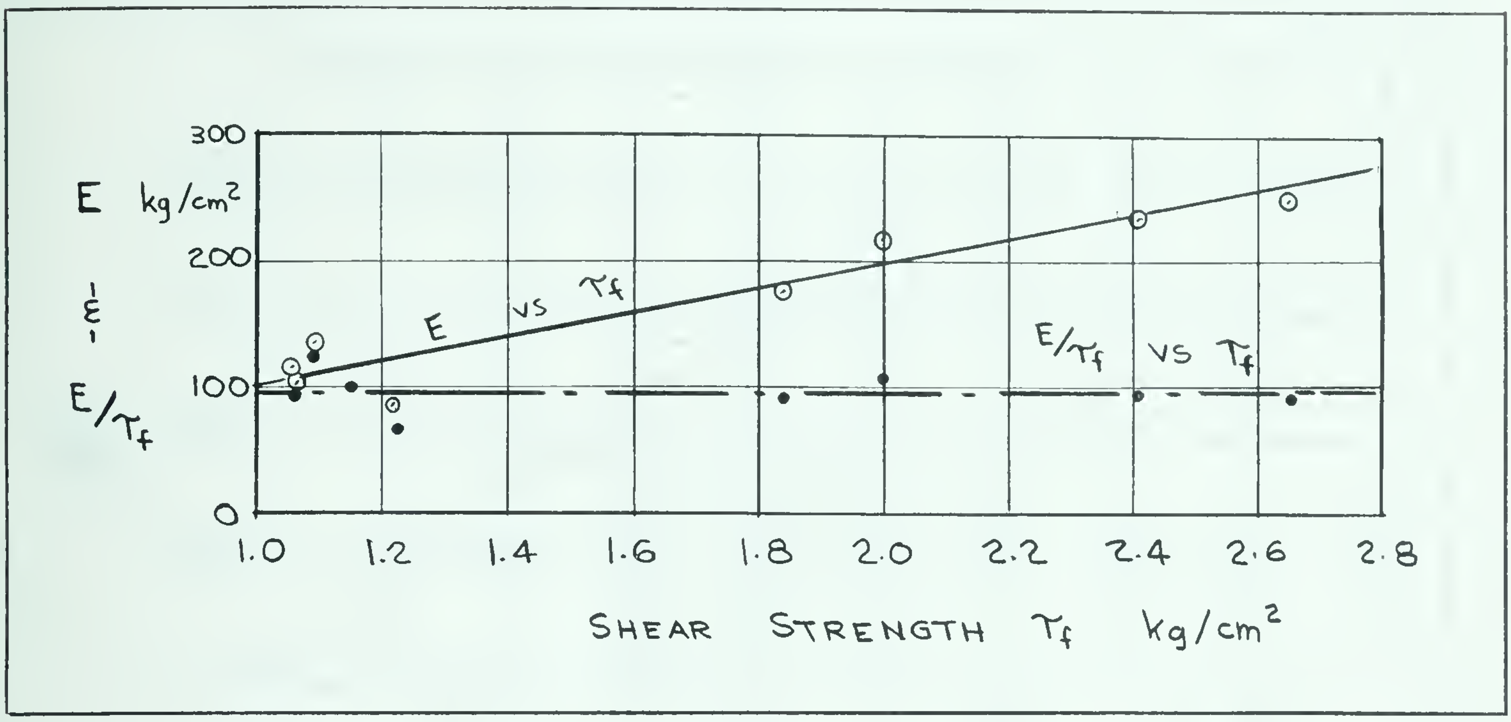


Figure 5-10 Variation of Secant Modulus of Elasticity E and Modulus Ratio E/τ_f, with Shear Strength τ_f.

5.6 Skin Friction Tests

In Table 5-2 and Figures 5-11 and 5-12 are given the results of the 5 direct shear tests on brass plates with A-4 shaft soil. Figure 5-11 shows the ratio of maximum skin friction to shear strength f_{max}/τ_f plotted against the normal stress. The ratio f_{max}/τ_f is the same as the symbol α for pile skin friction used in equation 3-7b, page 47. A conjectured failure envelope has been shown through the plotted points up to a maximum value at which the skin friction is assumed to be equal to the shear strength of the soil. The envelope is similar to the hypothetical relationship of Figure 3-1 (except that the ordinate is divided by τ_f) but with zero adhesion.

The stress-strain characteristics of the skin friction tests are shown in Figure 5-12, with relative skin friction f/f_{max} plotted against the surface movement between the

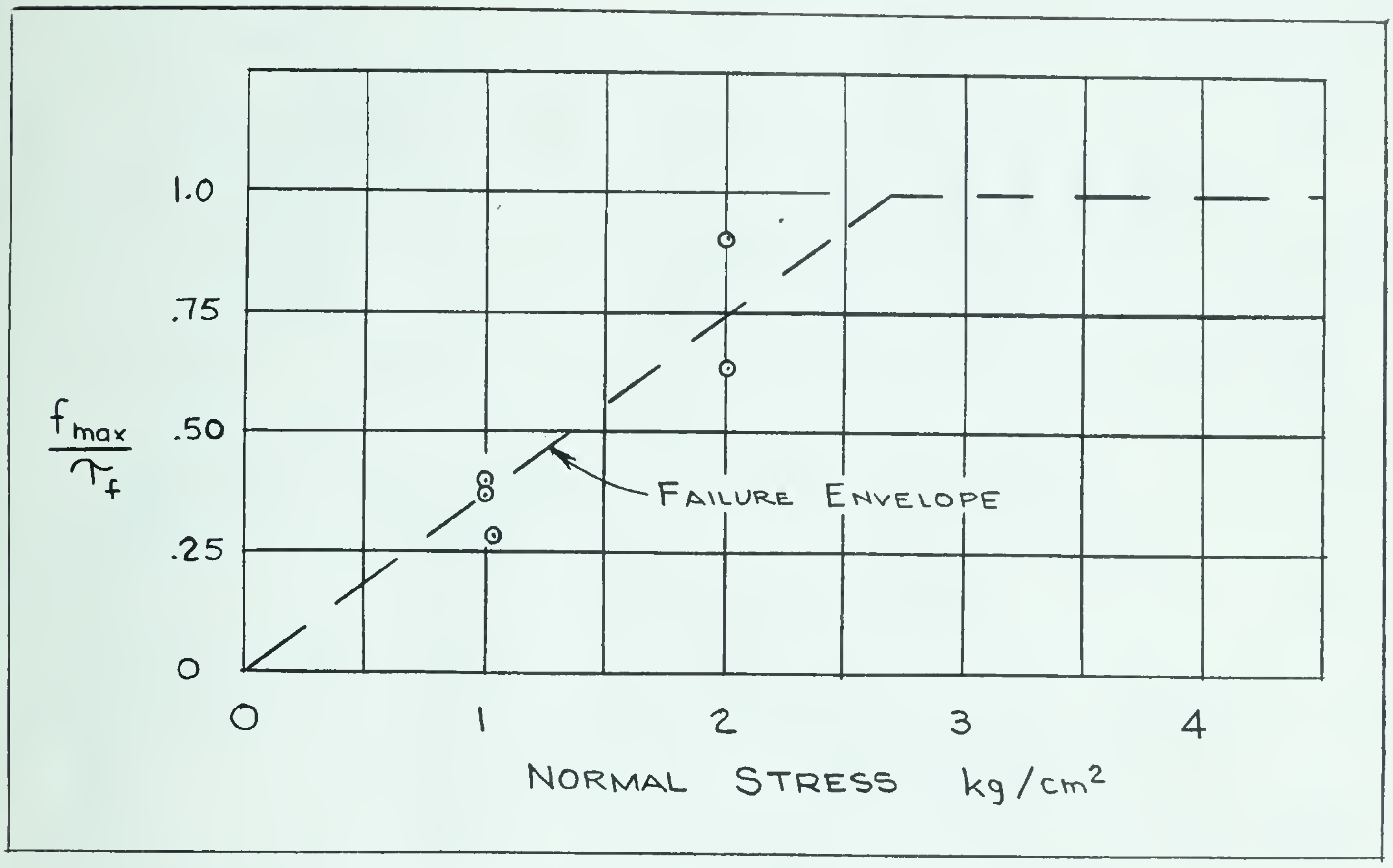


Figure 5-11 Direct Shear Test Results Brass Plates

TABLE 5-2

DIRECT SHEAR TESTS ON BRASS PLATES AND SOIL, TEST A-4

Test No.	Δ_{max} In.	Normal Load σ kg/cm^2	Maximum Shear stress f_{max} kg/cm^2	$\frac{f_{max}}{\tau_f}$	Remarks
F1 B1	.017	1.03	.265	.257	Tests in
F2-1-B1-1	.0235	1.00	.438	.400	direct shear
F2-1-B1-2	.0235	1.00	.409	.375	box on
F2-1-B2	.0388	2.00	.692	.635	smooth brass
F2-2-B1	.0424	2.00	.995	.913	plates. Area 36 cm^2 Plate .17cm thick.
F2-1-DS		1.01	1.000	.9174	Soil only

From Unconfined Compression Test τ_f = 1.09 kg/cm^2

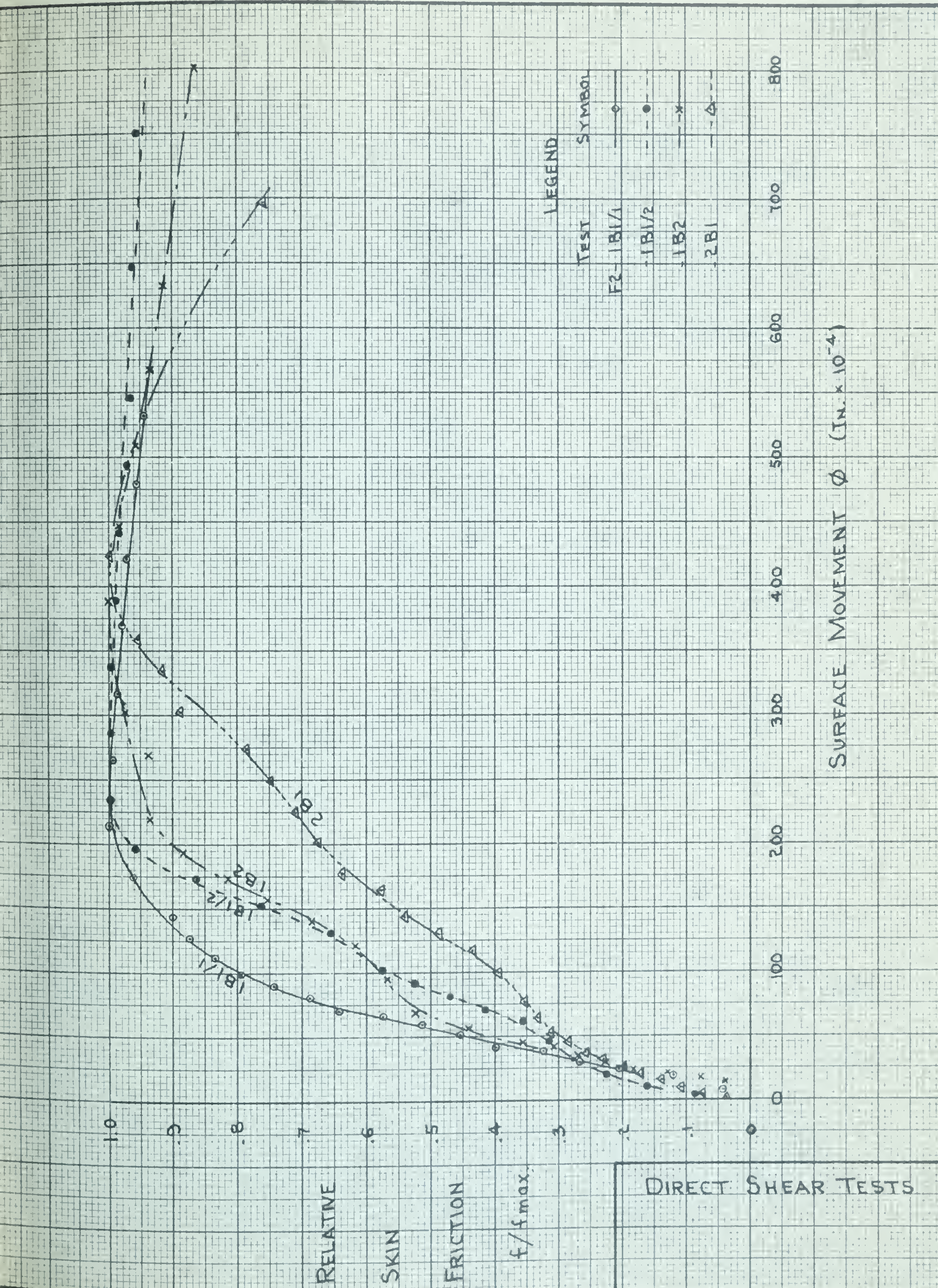


Figure 5-1[illegible] Relative Skin Fr[illegible]

brass plate and soil. The curves all demonstrate in varying degrees a step-like relationship between increasing stress and strain. The irregularity is in part a result of the strain mechanism (a geared linkage turned by a hand wheel) of the direct shear apparatus and possibly in part a result of a "stick-slip" phenomena which is a result of the difference between the static and kinetic friction of the surface (Horn, 1962, pg 324).

5.7 Computer Processing of Data

The University of Alberta IBM 1620 computer was used to analyse the results of the calibration and pile tests. Two basic programs were used, one to analyse the calibration test data, Routine B, and one to analyse the test data, for the pile in soil, Routine A. Both programs were written in the IBM Fortran language and processed through the FORGO Compiler.

Routine A was designed to evaluate the vertical and horizontal strain readings at each pile load in terms of the equations given in Chapter IV.

$$Q = A_c \, E_c \, \epsilon \qquad 4\text{-}1$$

$$P_h = \frac{2t}{b} \sigma_\theta \qquad 4\text{-}4$$

where $$\sigma_h = \frac{E_p}{1-\mu^2} (\epsilon_h + \mu \epsilon_\theta) \qquad 4\text{-}2a$$

and $$\sigma_\theta = \frac{E_p}{1-\mu^2} (\epsilon_\theta + \mu \epsilon_h) \qquad 4\text{-}2b$$

ϵ_h and ϵ_θ are the average strain readings in the vertical and circumferential directions. The program also evaluated the amount of compression of the pile shaft at each gauge level, the accumulative compression and the total

penetration of the pile at each level. The symbols used in the computer output together with all results are given in Appendix K.

Routine B was developed to analyse the data from the calibration tests where the pile was loaded free-standing. This was necessary because the presence of the rings inside the pile shaft introduced hoop stresses in the shaft which resulted in a fictitious lateral pressure on the pile (equation 4-4) even with the pile not surrounded by soil. The program calculated both Q_h and P_h at each gauge level and also the ratio of Q_h to the actual pile load Q_T, as measured by the proving ring. This ratio should be 1.0 of course, and is therefore a measure of the accuracy of the strain readings. Also computed by this routine at each gauge level was the average vertical strain; that is ϵ_h.

Routine B was further modified and applied to the pile test data to calculate the ratio of the measured vertical load Q_h at each gauge level in terms of the maximum failure load of the pile, Q_{max}. This was done to enable the results to be plotted non-dimensionally. This modified program was called Routine C.

5.8 Load Settlement Curves

A plot of total pile load, in kilograms, versus cap settlement divided by base diameter is given in Figures 5-13 and 5-14 for Tests A-1a and A-4, and A-2b and A-3 respectively. Tests A-1b and A-2a were not plotted because the results were not satisfactory. Tests A-1a and A-4, and A-2b and A-3 were grouped together because they represent similar situations with respect to soil strength. Tests A-2b and A-3 were tests in which shaft

and base soils were nearly the same strength, and tests A-1a and A-4 in soils where the base soil was stronger than the shaft soil.

In the subsequent analyses of the model pile tests their behaviour is compared with that of other model pile and field pile tests. The foregoing relationships between the shear strengths of the model pile shaft and base soils must be kept in mind when making these comparisons. In some cases the field pile tests are in heterogeneous soils with the shear strength nearly constant with depth, in other cases the shear strength increases with depth. For comparison purposes the important pile dimensions and average soil properties are listed in Table 5-3. It should be remembered that the field soil strengths shown are those representative of the area in which the test was made but do not reflect the influence which the installation method would have on the soil immediately adjacent to the pile.

In Figure 5-15 the load settlement curves for tests A-1a and A-2b are shown to exaggerated scale to illustrate their behaviour at small loads.

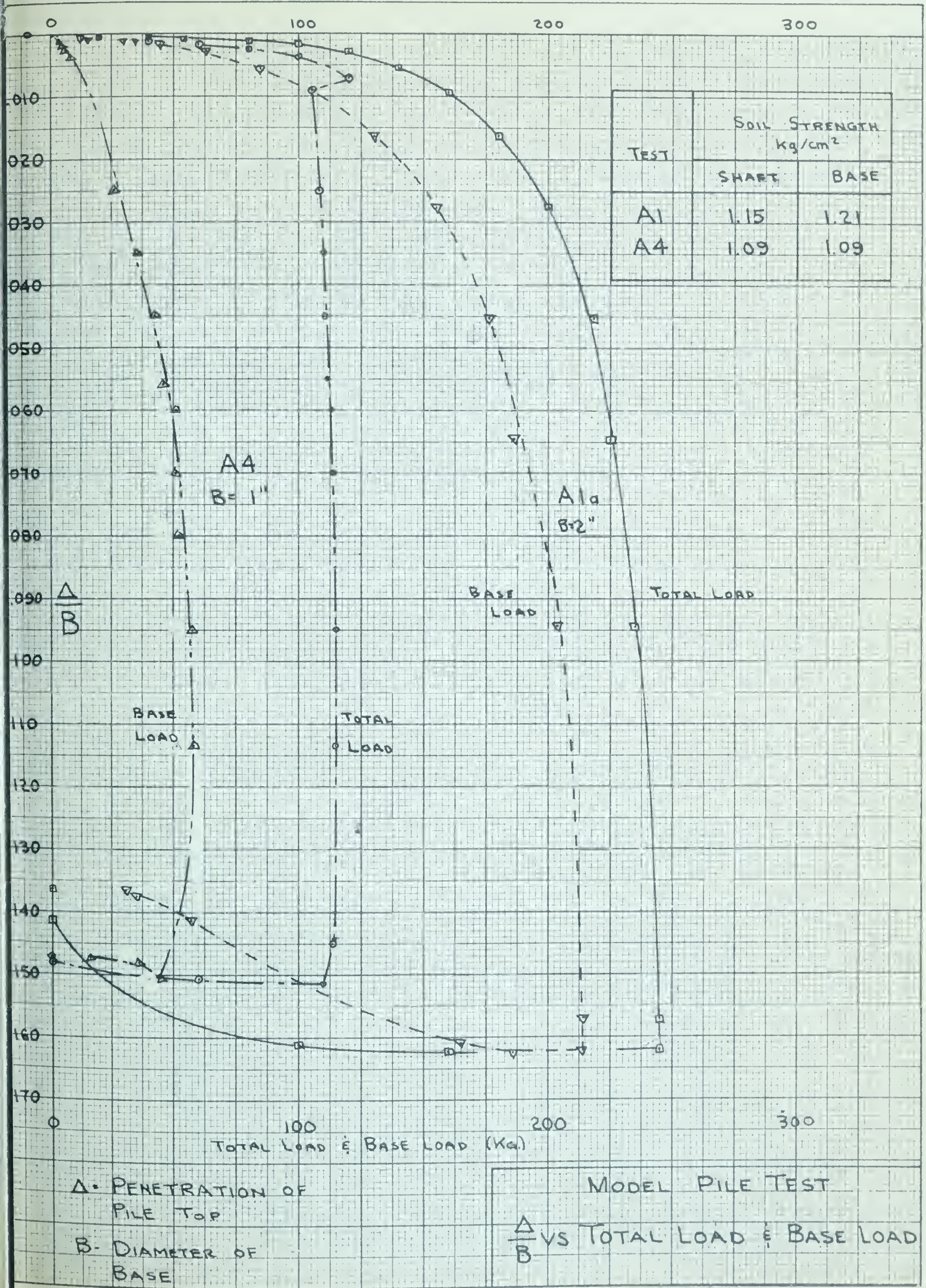
The curves for total load are typical of those for prototype field tests; a straight portion followed by a curving portion which becomes nearly assymptotic to the vertical at the failure load (Dubose, 1955, pg 158-159; Whitaker, 1962, pg 694). One advantage of model pile tests is illustrated here, in that it is possible to take the pile to its failure load (ie. complete rupture); in many field tests this is not possible because of the heavy loads required.

TABLE 5-3

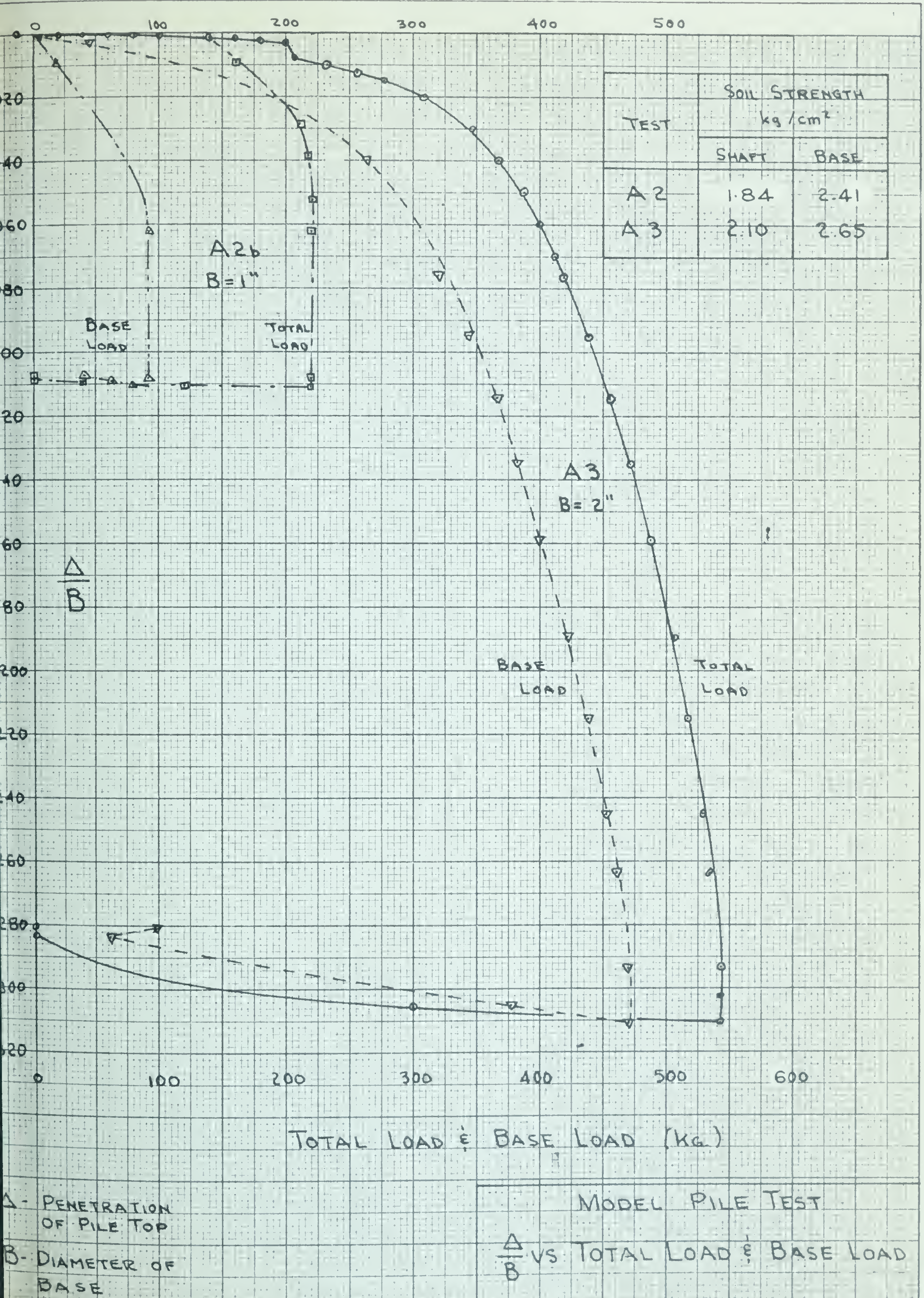
COMPARISONS OF MODEL AND FIELD PILE TESTS

Pile Test	Soil Undrained Shear Strength kg/cm^2 Shaft c_s	Base c_B	Ratio, c_s/c_B	Shaft diam In.	Base diam In.	Pile Depth D
Model Piles[1]						
A-1	1.15	1.21	0.95	1.0	2.0	11"
A-2	1.84	2.41	0.76	1.0	1.0	12"
A-3	2.10	2.67	0.79	1.0	2.0	11.9"
A-4	1.09	1.09	1.00	1.0	1.0	12.4"
Model Piles[2] (Cooke, 1961)	Both same (0.7 to .14)		1.0	.75	var.	var.
Model Piles[3] (Sowers, 1961)	0.8 psi	0.8 psi	1.0	1.21	1.21	29"
Cliffs-Pavillion[4] (Whitaker, 1962)	1.2	1.4	0.85	15	15	27"
St. Giles[5] (Frischmann, 1962)	1.85	2.85	0.65	36	73	70'
Jane Street[6] (Lo, 1964)	0.8	0.8	1.0	22	36 approx.	26'
Texas[7] (Dubose, 1955)	0.4	0.4	1.0	7	7	6' approx.

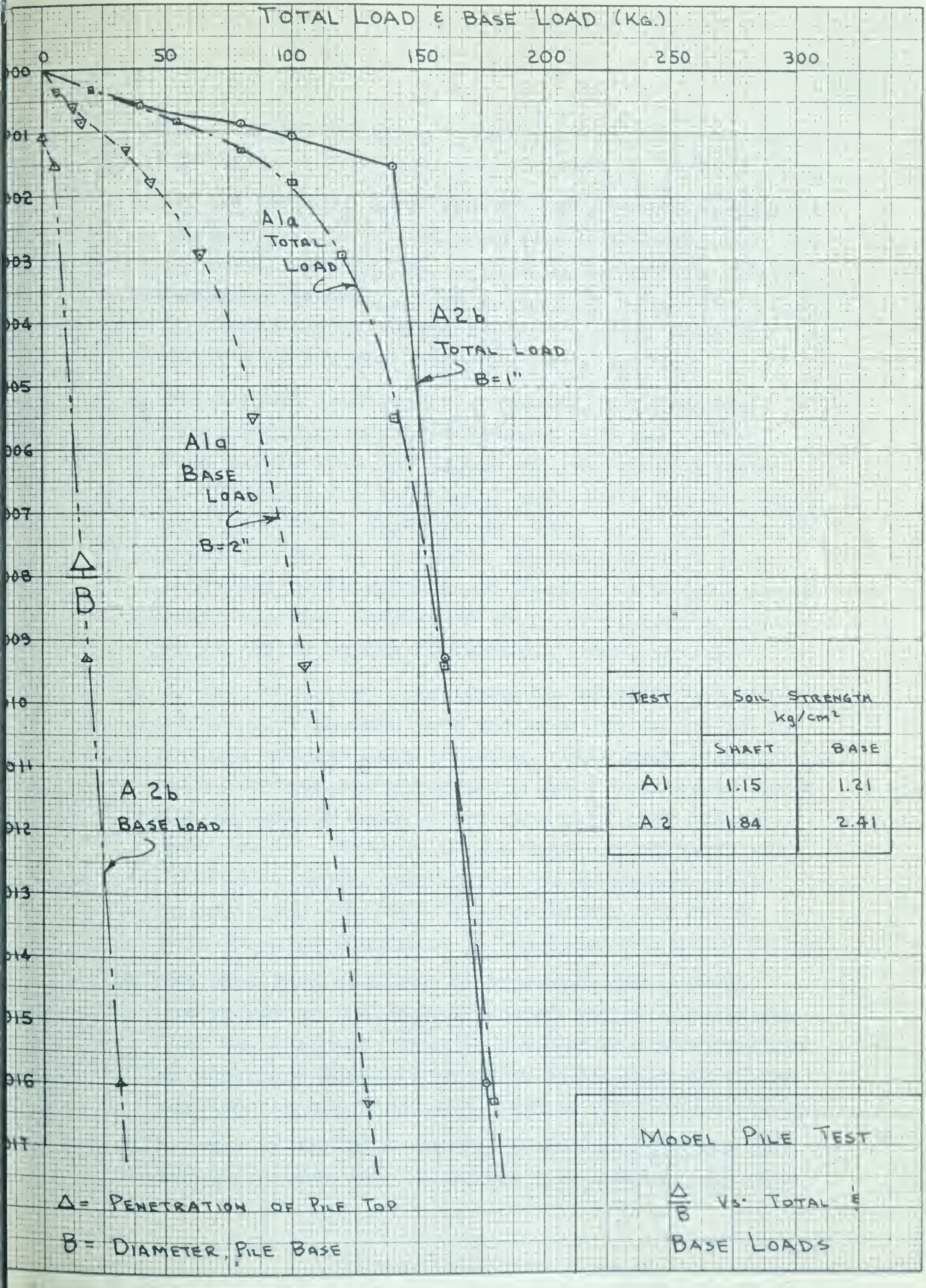
1. Model piles in compacted clay-this thesis
2. Model piles in remoulded brown London clay
3. Model pile in remoulded commercial bentonite
4. Drilled, cast-in-place concrete pile in brown clay
5. Drilled, cast-in-place concrete pile in London clay
6. Franki displacement concrete caisson in silty-clay with sand around upper 12 feet.
7. Drilled, cast-in-place concrete pile in clay.



0
100
200
300
SOIL STRENGTH kg/cm²
TEST
SHAFT
BASE
A1
1.15
1.21
A4
1.09
1.09
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.110
.120
.130
.140
.150
.160
.170
Δ/B
A4
B = 1"
A1a
B = 2"
BASE LOAD
TOTAL LOAD
BASE LOAD
TOTAL LOAD
TOTAL LOAD & BASE LOAD (Kg.)
Δ - PENETRATION OF PILE TOP
B - DIAMETER OF BASE
MODEL PILE TEST
Δ/B VS TOTAL LOAD & BASE LOAD



Soil Strength kg/cm²
Test
Shaft
Base
A 2
1.84
2.41
A 3
2.10
2.65
A2b
B = 1"
Base Load
Total Load
A3
B = 2"
Base Load
Total Load
Δ/B
Total Load & Base Load (kg)
Δ - Penetration of Pile Top
B - Diameter of Base
Model Pile Test
Δ/B vs Total Load & Base Load


TOTAL LOAD & BASE LOAD (KG.)
0
50
100
150
200
250
300
A1a TOTAL LOAD
A2b TOTAL LOAD B=1"
A1a BASE LOAD B=2"
Δ/B
A 2b BASE LOAD
TEST
SOIL STRENGTH kg/cm²
SHAFT
BASE
A1
1.15
1.21
A 2
1.84
2.41
Δ = PENETRATION OF PILE TOP
B = DIAMETER, PILE BASE
MODEL PILE TEST
Δ/B Vs. TOTAL & BASE LOADS

The observations which may be made on these curves are as follows:

1. At the "working load" for the model piles (approximately 1/3 of the ultimate load) between 75% to 90% of the load is carried by the shaft. This compares favourably with a value of about 80% for a prototype cast-in-place test in a stiff clay at Cliff's Pavillion (Whitaker, 1962, pg 694).

2. The maximum shaft load is reached at relatively small penetrations (Δ/b between .47% and .93%). This agrees with the values of .5% found by Cooke and Whitaker for both model pile tests and the above mentioned field test (Cooke, 1961, pg 7 and Whitaker 1962, pg 694) but not with the results of a test performed by Frischmann and Fleming in which maximum shaft load was reached at 6% in stiff London clay at St. Giles Circus (Frischmann, 1962, pg 129).

3. In all four model pile tests the average skin friction at the pile failure load was only 45% to 90% of the maximum average skin friction. This same drop-off in skin friction was observed in the direct shear tests and in work on model piles in a soft clay (Cooke, 1961, pg 7). However in the tests by Whitaker and Frischmann quoted previously and by Dubose (1955, pg 159), all for cast-in-place piles, the shaft skin friction continued to increase with penetration.

4. The base load required considerable penetration (6% to 30% of the base diameter) before its maximum load was reached. This is similar to the experience of Cooke and Whitaker in their model pile tests and Whitaker in his field pile test at Cliff's Pavillion.

5. Rebound of the pile cap is almost entirely due to rebound of the soil below the base and in fact the

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

shaft soil resists this rebound. Frischmann reports this same effect for stiff London clay (1962, pg 130).

6. The sometimes erratic shape of load settlement curves, such as illustrated by Test A-4 may be due to the decrease in shaft load after failure (and before the base load builds up to its maximum value). This particular behaviour was very noticeable in the model pile tests conducted by Cooke and Whitaker.

5.9 Skempton's N_c and α Parameters

The empirical formula developed by Skempton on the basis of many field tests on cast-in-place piles in London clay was given in section 3.6 and is repeated here:

$$Q = c_B \cdot N_c \cdot A_B + \alpha \cdot c_s \cdot A_s \tag{3-8c}$$

The parameter N_c is the same as Meyerhof's general bearing factor N_{cqr} and has a value nearly 9.0. The parameter α is a measure of the average amount of the shear strength of the soil actually developed in skin friction and for London clays was found to be very nearly 0.45.

The N_c and α parameters for the model pile tests are given in Table 5-4. The parameters were also calculated at the penetration at which the α coefficient is a maximum. Comparisons of the results show clearly that the maximum skin friction is much greater than the ultimate or final skin friction and that at maximum skin friction the base resistance is very low.

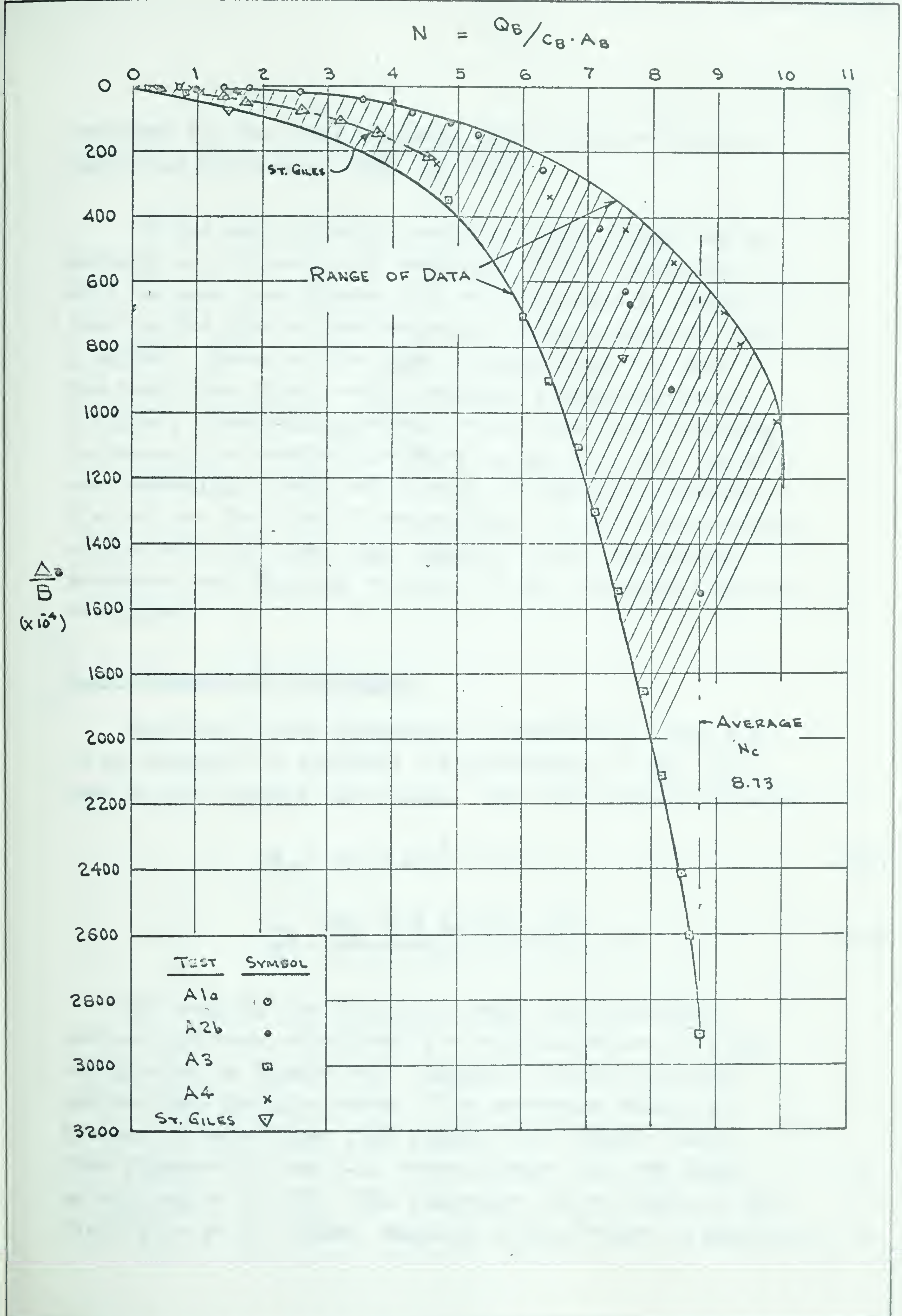
TABLE 5-4

SKIN FRICTION AND BEARING FACTORS

Test	Δ_{max}	B	$(\frac{\Delta}{B})_{max}$	$\Delta\alpha_{max}$	α		N_c	
					α_f	α_{max}	$(N_c)_f$	$(N_c)\alpha_{max}$
1a	.3140	2	.1570	.0059	.135	.248	8.73	2.57
2b	.0623	1	.0623	.0093	.309	.345	7.53	1.48
3	.5876	2	.2938	.0047	.167	.372	8.75	0.34
4	.0950	1	.0950	.0070	.224	.419	9.92	2.42

The average bearing capacity factor N_c is 8.73, comparing closely with the value given by Skempton and others (see Table 2-1 page 21). The range of values from all tests is illustrated by Figure 5-16 which shows the ratio N, defined as the ratio of base load to base soil shear strength Q_B/c_B, plotted against the base penetration ratio Δ_B/B. At its maximum value N is equal to N_c. It can be seen that while the settlement curves differ appreciably, they are more or less asymptotic to a value of N_c of about 9.0. As an illustration of the settlement curve of the base of a field pile the data for a test at St. Giles, England in which base load and settlement were measured (Frischmann, 1962, pg 124 & 128) is also plotted on the same figure.

The average value of α at ultimate or final load is only 0.209, about one half of the value given by Skempton, although values as low as .3 and .38 are



N = QB/cB·AB
0 1 2 3 4 5 6 7 8 9 10 11
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
ΔB/B (x10⁻⁴)
ST. GILES
RANGE OF DATA
AVERAGE Nc 8.73
TEST
SYMBOL
A1a
A2b
A3
A4
ST. GILES

reported for the cast-in-place field piles at Cliff's Pavillion (Whitaker, 1962, pg 694).

If the skin friction tests on brass plates can be assumed to represent the behaviour of the model pile it will be seen from Figure 5-11 that the average normal load on the pile at maximum skin friction would be about 1 kg/cm^2. Since at the time of compacting the clay in the mould the final static vertical stress was about 5 kg/cm^2, considerable stress relief must have occurred to reduce the vertical pressure on the pile to 1 kg/cm^2, even assuming a ratio of lateral to vertical pressure, $k_o = 0.5$, at the time of compaction. In fact some rebound of the soil did occur upon removal of the compacting pressure, and this may account for the low skin friction developed.

5.10 Estimate of Settlement

From the theory presented in Sections 3-7 and 3-8 it is possible to estimate the settlement of the pile base Δ_B for various base loads. The basic equations are:

$$N_c = 4/3\ (\ln \frac{E}{c} + 1) + 1 \qquad 2\text{-}17$$

$$\frac{\Delta_B}{B} = \frac{N_c \cdot g\ M\ I_\infty (1 - \mu^2)}{2} \cdot \epsilon \qquad 3\text{-}26$$

For each of the four pile tests the settlement ratios Δ_B/B were calculated for various ratios of q/q_f and plotted on Figure 5-17 together with the observed points from the pile tests. The necessary values of E/c and ϵ were taken from Figure 5-9, M and g taken from Figures 3-8 and 3-11 respectively; I_∞ was taken as $\pi/4$ and μ as 0.5. The behaviour for the base of the field pile at St. Giles, England is also shown on Test A-1.

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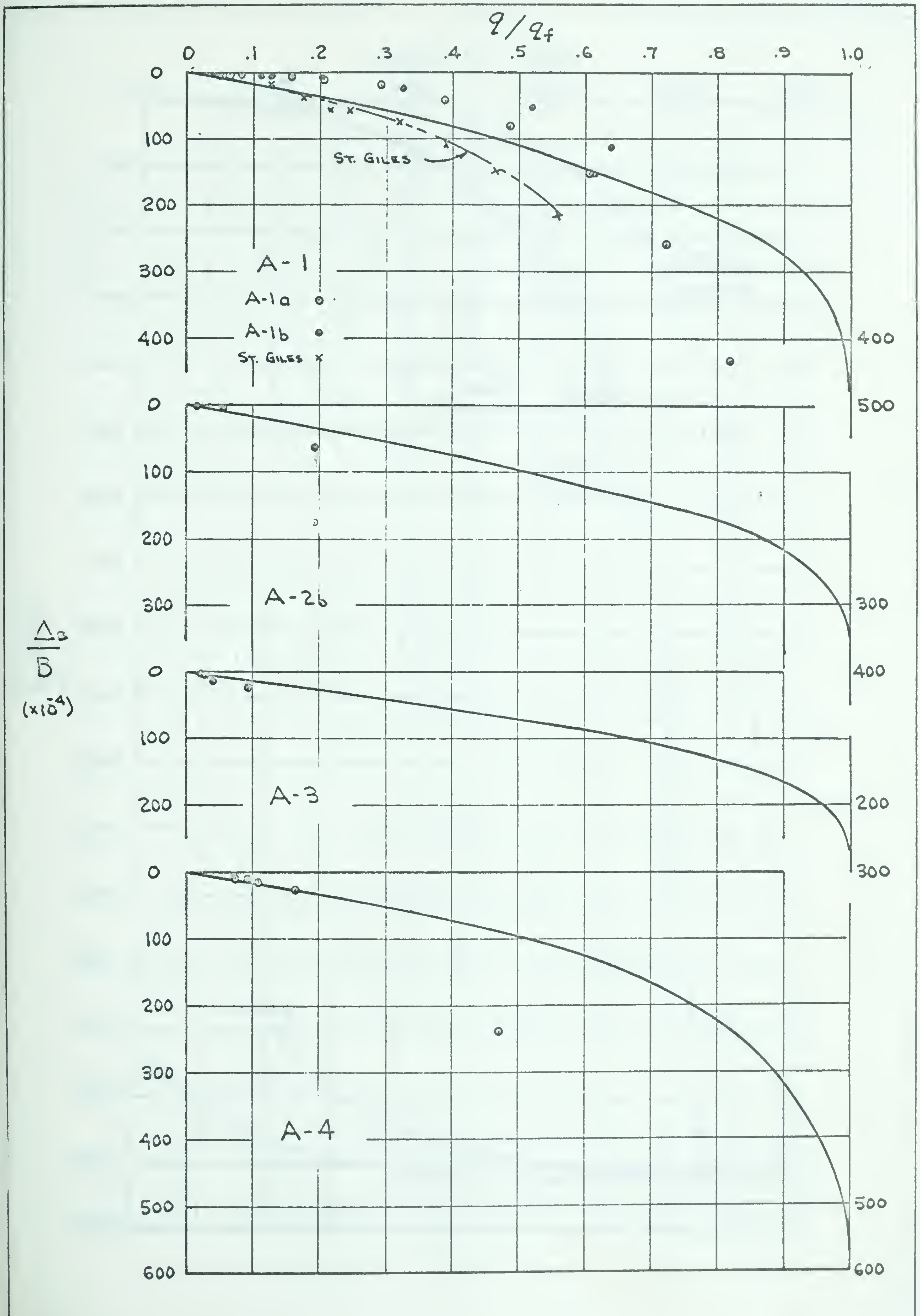

Figure 5-17 Base Load Ratio q/q_f Plotted Against Settlement Ratio $\Delta_B/$

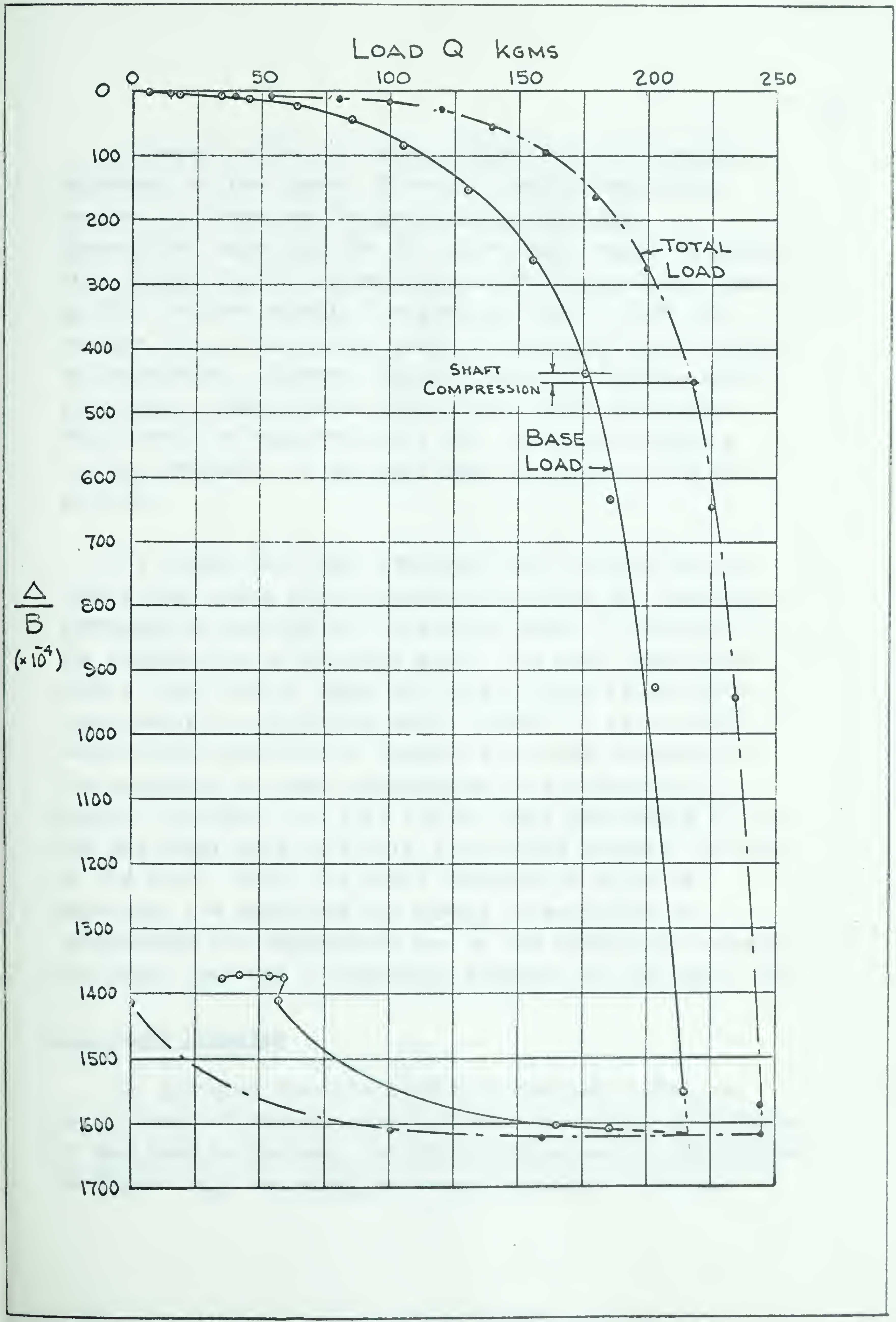

Figure 5-18 Load Settlement Diagram, Test A-1a

Except for Test A-1 the settlements are in broad agreement at low loads. There is insufficient data, however, to draw any far-reaching conclusions. A theoretical curve for the St. Giles test cannot be given even though the E/c is known to be 65 (Frischmann, 1962, pg 129), almost exactly the same as that of Test A-1, because the stress-strain curve for the soil is not given by Frischmann. However the settlement of the St. Giles pile base is about twice that of the model pile base which would be expected since the correction factor g for the proximity of the pile base to the mould ($\frac{Z}{B}=2.5$) is 0.58.

To obtain the total settlement at the pile cap by theoretical means it is necessary to solve the equations presented in section 3-7 or similar ones, to account for the compression of the pile shaft. In many cases where much of the load is taken by the pile base (piles with large bells or relatively short shafts) it is probably sufficiently accurate to neglect the shaft compression. The magnitude of shaft compression is illustrated in Figure 5-18 where the pile cap and base settlement ratios for the model pile test A-1a are plotted against the load on the pile. Where the shaft compression might be important its magnitude can always be estimated by calculating the compression due to the difference between the total load and a reasonable estimate of the base load.

5.11 Load Transfer

By plotting the pile load Q at various depths, h, or in terms of dimensionless ratios, the effect of transfer of the load to the soil in skin friction can be illustrated. In Figure 5-19 two examples are given; Test, A-2a and

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A-2b, with the pile load Q in kilograms plotted against the ratio of depth to shaft length h/H. For purposes of comparison, the results of all tests are plotted in Figures 5-20 to 5-22, this time using the dimensionless ratio Q/Q_{max} to represent the pile load. Because of considerable scatter of the data the curves are necessarily interpretive. The proving ring load and base load as determined by the load cell were taken as being the most accurate of the readings and used as fixed points in drawing the curves.

For a uniform distribution of skin friction the lines will be straight. The magnitude of skin friction is inversely proportional to the slope of the line. The results show that at low loads the lines are reasonably straight and decrease their slope with higher pile loads. For higher loads, above about 30% of the ultimate load Q_{max}, this pattern does not exist for all tests, and in Tests A-1b, A-2a and A-4 the skin friction is definitely not uniformly distributed.

The magnitude of skin friction at any depth may be determined from the slope of the load-transfer curve using the relationship of equation 3-18b

$$\frac{dQ}{dh} = f \cdot C \qquad \text{3-18b}$$

or in terms of the dimensionless ratios h/H and Q/Q_{max}

$$f = \frac{dQ'}{dh'} \cdot \frac{Q_{max}}{CH} \qquad \text{5-2a}$$

where dQ'/dh' is the slope of the line in Figure 5-20 to 5-22. For the model pile tests with the circumference

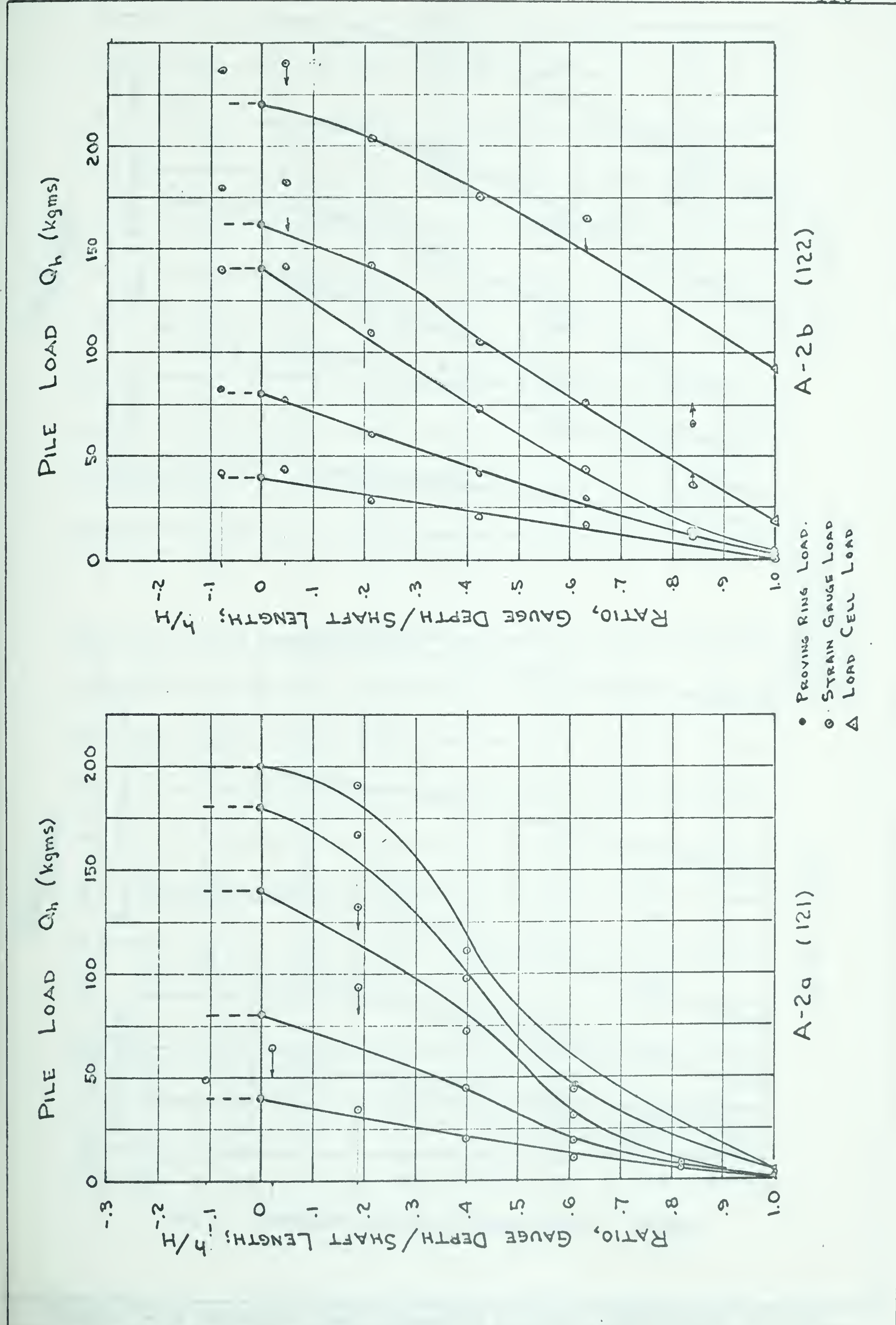

Figure 5-19 Load Transfer Curves, Tests A-2a & A-2b

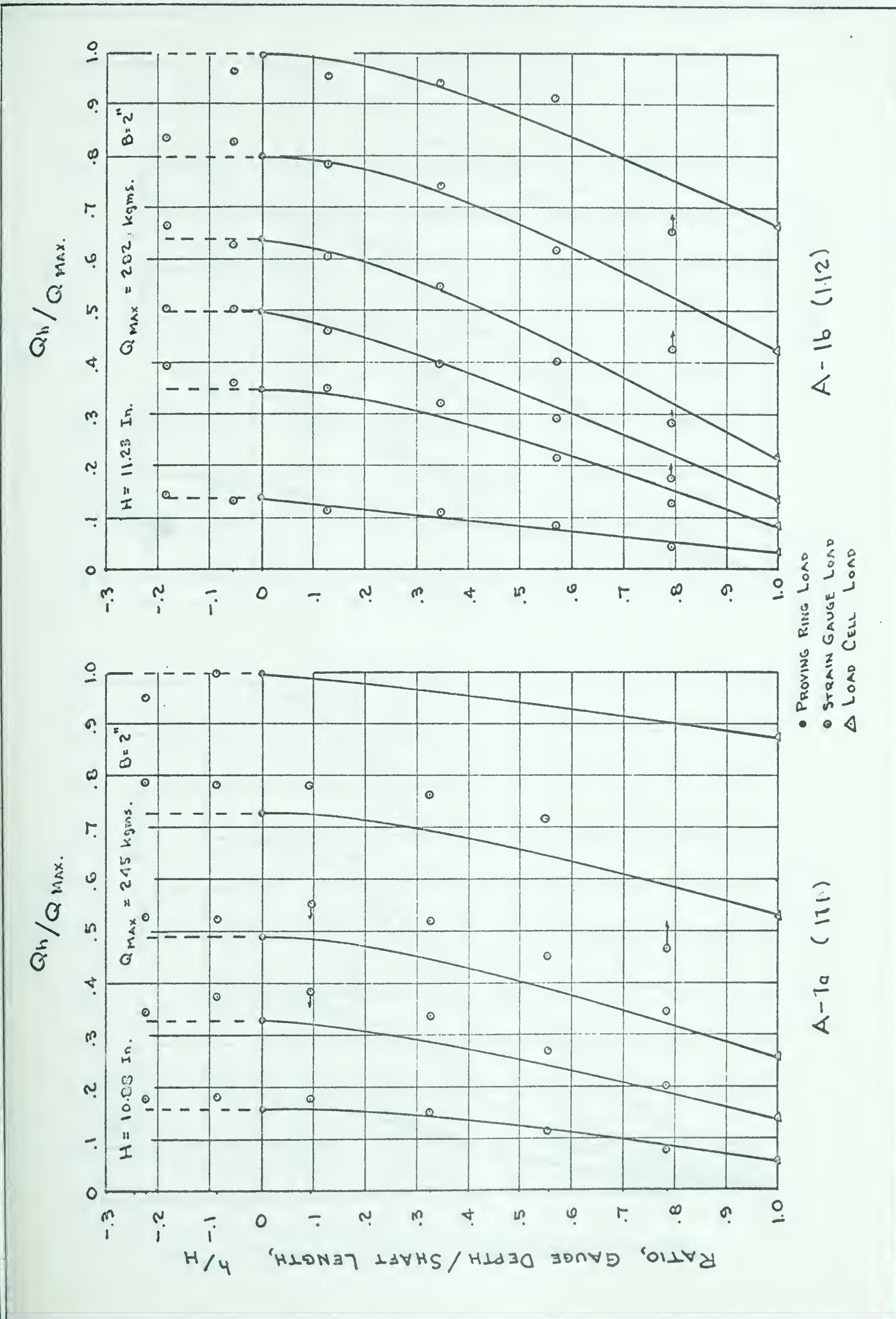

Figure 5-20 Relative Load Transfer Curves, Tests A-1a & A-1b

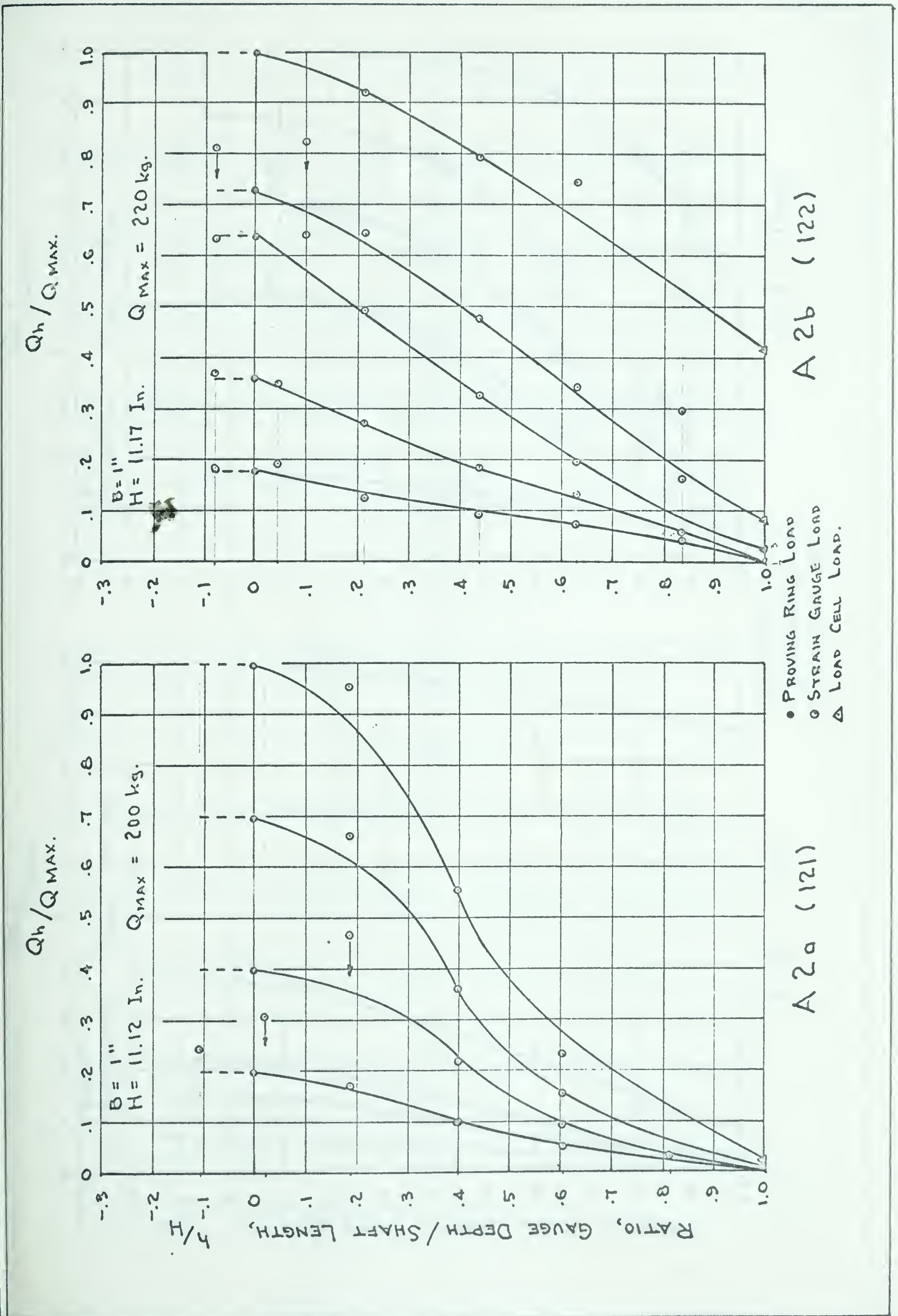

Figure 5-21 Relative Load Transfer Curves, Tests A-2a & A-2b

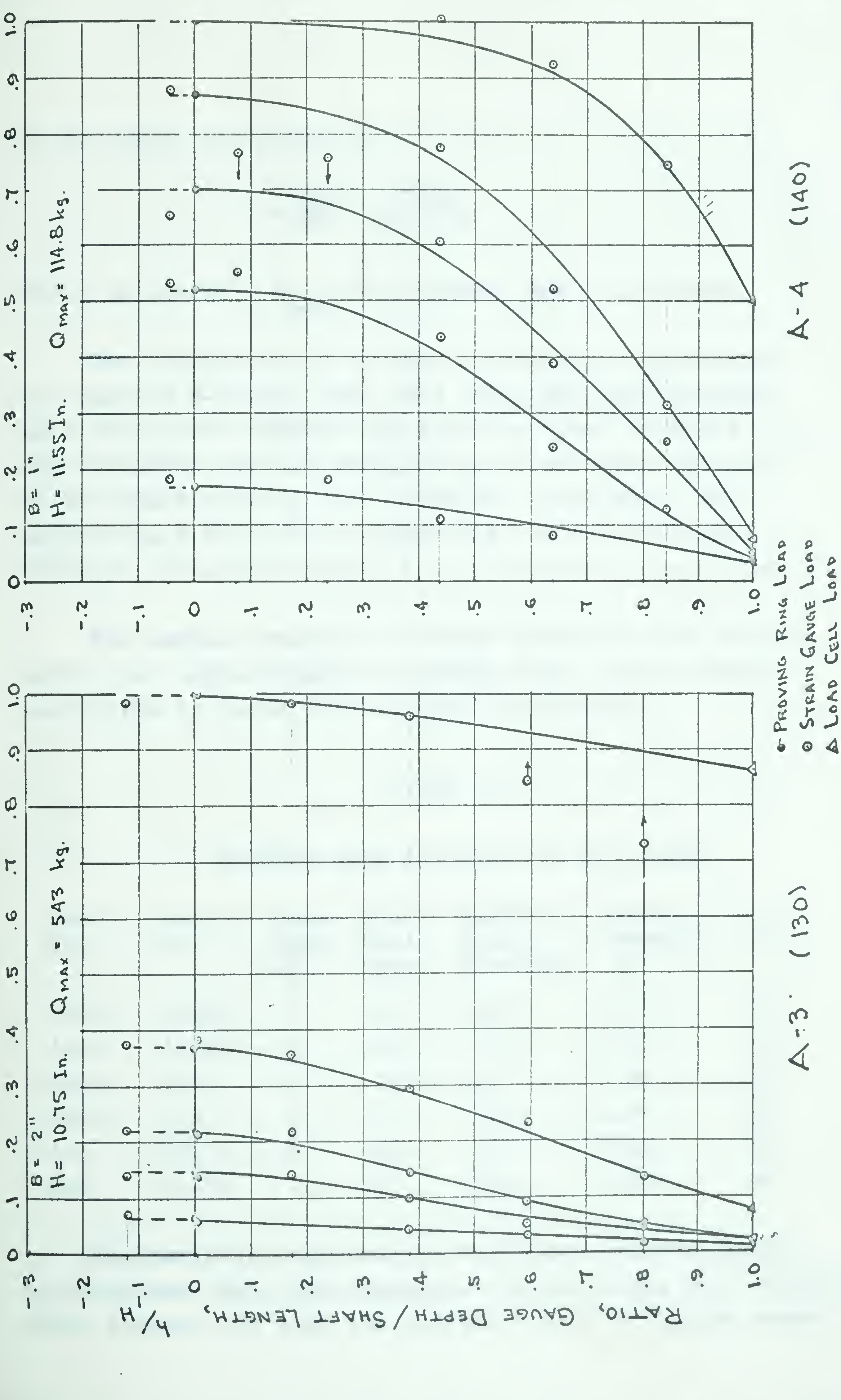

Figure 5-22 Relative Load Transfer Curves, Tests A-3 & A-4

of the shaft C = 8.011 cm.

$$f = \frac{dQ'}{dh'} \cdot \frac{Q_{max}}{20.3\ H} \qquad 5\text{-}2b$$

for f in kgs/cm^2, Q_{max} in kilograms and H in inches.

The maximum value of skin friction was calculated by equation 5-2b for each test using the load transfer line having the maximum slope and is shown in Table 5-5. Also recorded are the average undrained shear strength of the shaft soil τ_f, the depth of shaft where the maximum skin friction occurred and the ratio of skin friction to shear strength f/τ_f (Skempton's coefficient α).

The maximum values of α found from the load transfer curves are higher than the average value for the same test given in Table 5-2, as would be expected.

TABLE 5-5

MAXIMUM SKIN FRICTION ON PILE SHAFT

Test No.	Depth In.	Base Diam. In.	Shaft Load kgms.	Maximum Skin Friction	Shear Strength τ_f	$\alpha = f/\tau_f$
A-1a	10.88	2	64	.334	1.15	.30
A-1b	11.23	2	62	.744	1.15	.65
A-2a	4.0	1	128	1.77	1.84	.96
A-2b	5.6	1	92	.776	1.84	.42
A-3	10.75	2	41	.747	2.00	.37
A-4	11.55	1	57	.881	1.09	.81

The variable behaviour of the load-transfer curves is consistent with the appearance of the model pile surface after failure, in that the surface showed irregular areas

of soil adhering to the shaft and which had apparently failed in shear, see Figure 5-1. There seems to be no adequate explanation for the cause of these irregularities either in terms of the size of the base, relative strengths of the shaft and base soils or in terms of the vertical stress illustrated in Figures 3-9 and 3-10.

Except for pile Test A-2 the load transfer curves show that the maximum load transfer or skin friction occurs near the base of the pile. This result is similar to that found by Sowers et al for model pile tests in a homogenous clay of shear strength of only 0.8 lbs/sq in. (Sowers, 1961, pg 156).

The results in Table 5-5 also indicate that the value of α for maximum skin friction is higher for the 1 inch diameter base than for the 2 inch diameter base. The explanation of this behaviour may lie in that larger tensile (or less compressive) stresses result from the larger base, in accordance with the stress distribution shown in Figure 3-9, pg 59.

It was initially assumed that the distribution of load between the shaft and base would be in part a function of the relative stiffness of the shaft and base soils. In Table 5-6 the results of the four tests are given in descending order of the ratio of the secant modulii of elasticity of the shaft and base soils (E shaft/E Base). Shown are the base diameters B, maximum shaft load Q_s, ratio of shaft to total load Q_s/Q_{max} at failure , ratio of maximum shaft load to maximum base load Q_s/Q_B and the ratio of skin friction coefficient to the general bearing factor α/N_c. Since this latter parameter is independent of pile dimensions it should

reflect the influence of the relative stiffness of the shaft and base soils on the load carried by shaft and base.

TABLE 5-6

VARIATION OF SHAFT LOAD WITH RELATIVE SOIL STIFFNESS

Test	Base Diam.	$\frac{E_{shaft}}{E_{base}}$	Q_s kgms	Q_s/Q_{max} %	Q_s/Q_B	α/N_c
A-1a	2	1.34	31	12.7	.145	.0156
A-4	1	1.31	57	49.9	.995	.0222
A-3	2	.88	73	13.5	.156	.0193
A-2b	1	.75	128	58.2	1.39	.0413

Inspection of Table 5-6 shows no particular relationship between the relative stiffness as measured by secant modulus of elasticity of the shaft and base soils and the distribution of load between base and shaft.

5.12 Lateral Shaft Pressures

Theoretically the results of the strain gauge readings at a given depth should give the lateral pressure on the pile, by using equations 4-2b and 4-4:

$$\sigma_\theta = \frac{E_p}{(1-\mu^2)} \left(\epsilon_\theta + \mu\epsilon_h \right) \qquad 4\text{-}2b$$

$$P_h = \frac{2t}{b} \sigma_\theta \qquad 4\text{-}4$$

The computer program was set up to give the lateral pressure at each gauge depth using the above equations.

However the results were extremely erratic due mainly to the restraint on the shaft walls by the load cell ring, loading cap and shaft rings. It was thought that the effect of the restraints could be eliminated by determining the fictitious lateral pressure in the calibration tests and adjusting the test results accordingly. However there was erratic behaviour for these readings as well which would make any analysis extremely suspect.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Restatement of the Problem

On the basis of present-day theory and experimentation it is possible to predict the ultimate load capacity of a single pile in a purely cohesive soil with some degree of certainty. The problem still facing foundation engineers is the selection of a working load which will not produce damaging differential or total settlements. To do this requires a knowledge of the load-settlement properties of the pile, and for large piles this can frequently only be obtained, at least for preliminary designs, from estimates based on the stress-strain properties of the pile and soil.

The purpose of this thesis has been to explore, by means of a model pile, the load-settlement relationships of a pile in a purely cohesive soil.

6.2 Parameters Studied

A review of literature on the subject of pile behaviour indicated that the load settlement characteristics of a pile would likely depend upon the undrained shear strength of the shaft and base soils, the dimensions of the pile and the stress-strain properties of the pile and soil.

The pile tests were designed to provide load-settlement relationships in which two sizes of pile base were used and in which the soil properties were varied by compacting

the soil to different densities and water contents.

The stress-strain properties of the soil and pile material were measured by cylindrical compression tests. For the final test the stress-strain behaviour of the pile shaft in skin friction was measured by a direct shear test of the soil compacted onto brass plates. The pile settlement was determined for maintained loads at low settlements and at a constant rate of penetration for the ultimate load.

In the analysis of the model pile test results comparisons have been made wherever possible with the results of other model pile tests and field pile tests. Except for the low average skin friction developed on the smooth pile surface the tests are in general agreement with other pile tests and with qualitative observations found in the literature.

The reason for the low skin friction may be partly due to the smoothness of the pile surface and partly due to the fact that because of the short length of the pile the effect of gravity is negligible in causing lateral pressures on the side of the pile. The inability of a small scale test to duplicate the effect of gravitational forces is one of the most serious defects of model pile testing. This appears to be true even in the case of purely cohesive soils whose shearing resistance is independent of the magnitude of the total stresses at failure.

6.3 Conclusions

The following conclusions are based on the results of the model pile tests:

1. They are believed to be in general agreement with

other model and field pile tests, particularly for cast-in-place belled piles in saturated or near saturated, stiff clays.

2. The shaft load builds up to its maximum value at penetrations of about .5% of the shaft diameter and then decreases to a lower value, between 45% and 90% of the maximum, at the ultimate pile load.

3. The base load increases to a maximum value at relatively large penetrations, in the order of 6% to 30% of the base diameter. In all cases the maximum base load occurred at maximum pile load.

4. The ultimate or final average skin friction of the soils tested (undrained shear strengths between 1.0 and 2.0 kg/cm^2) is between 25% and 42% of the undrained shear strength of the soil adjacent to the shaft.

5. Based on load transfer data along the pile shaft the maximum skin friction is greatest near the base of the pile and is in some cases between 65% and 96% of the undrained shear strength of the soil.

6. The distribution of load between the shaft and the base appears to depend much more upon the diameter of the base than upon the relative stiffness of the shaft and base soils as determined by the ratio of their secant modulii of elasticity.

7. The load settlement curve of a pile base can be predicted by the theory of elasticity with a fair degree of accuracy up to about the working load of the pile by using the actual stress-strain curve of the base soil. For total settlement at the pile cap the shaft compression must alos be added.

6.4 Recommendations

The results of any model test can only be confirmed by the performance of the prototype in the field. The advantages of laboratory model tests using closely controlled soil gradations and strength properties are offset by the inability to duplicate both scale and gravity effects and at the same time simulate construction procedures. The model pile tests therefore must be thought of only as a preliminary to further research if conclusive evidence is to be obtained on pile behaviour. The following recommendations are suggested as one approach to amore rational design basis for settlement estimation:

1. It is clear that before pile behaviour in stiff cohesive soil is understood a better understanding of skin friction must be obtained. Skin friction tests by vane shear or direct shear tests provide only empirical methods of estimating shear strain, because true shear strain is measured in angular rotation and not by surface movement. Tests should be carried out using different surface materials and soils, and in such a way that shear strains at the failure surface are measured. The texture, permeability, surface roughness and stress-strain properties of the surface materials should be measured quantitatively.

2. Larger scale model piles should be constructed and tested in a large testing pit, using essentially the same methods as used herein. One pile could be of aluminium and another of concrete; both should be fully instrumented to measure pile load, load transfer along the shaft and base load. The strain gauges used should be compatible with a multi-channel data processer so that almost instantaneous readings of loads could be taken even during a constant rate of penetration test.

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The possibility of using glued-metal joints for the aluminium (or other metal) pile should be considered. The instrumentation of the concrete pile would serve as a preliminary study for full-scale field tests on concrete piles.

3. Based upon the results of these investigations, and of course on concurrent work by others, at least two fully-instrumented field pile tests should be carried out. One pile should be tested to failure, and the other, of identical dimensions and in identical soil if possible, should be designed as a load-carrying pile in a building. The piles should be instrumented to measure pile load, load transfer along the shaft and base load. Although nearly all strain gauges suffer from creep effects one of the purposes of the building pile should be to examine the change in shaft and base loads with time. An ideal location for such piles would be in the proposed Engineering Complex at the University of Alberta.

4. Any measurement of load transfer along the shaft must be made in terms of the elastic behaviour of the pile. For hollow sections the strain gauges must measure both vertical and horizontal strains.

5. Since the soils which most closely approximate a quasi-elastic material are the saturated or highly impermeable clays, it is recommended that further research be concentrated on this type of material for investigation of load settlement relationships.

6. Any full scale tests should in addition to measuring soil parameters by triaxial compression and direct shear tests also incorporate in-situ tests such as the static cone penetrometer, vane shear and possibly

even small scale plate bearing tests. In particular the static cone penetrometer has been used very successfully to predict pile capacities and might be used to predict pile settlements as well.

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APPENDICES

APPENDIX A

LAKE EDMONTON CLAY PROPERTIES

APPENDIX A

LAKE EDMONTON CLAY PROPERTIES

1. Origin — The Lake Edmonton Clays were formed in glacial times by sedimentation of clays in Glacial Lake Edmonton. (Bayrock and Hughes, 1962, pg 13)

2. Source — Material for test taken from a depth of approximately 6 feet from surface south side of a road cut in Belgravia Ravine, Edmonton, Alta.

3. Engineering Properties:

Test	Value
a. Specific gravity	2.73
b. Grain size distribution	
% sand sizes ($>$ .06 mm)	2
% silt sizes (.06-.002mm)	45
% clay sizes ($<$ .002mm)	53
c. Atterberg Limits	
Liquid limit W_L	76.90%
Plastic limit W_p	27.50%
Plasticity Index I_p	49.40%
Shrinkage Limit W_s	15.0%
d. Activity $A = \frac{I_p}{\%C}$	0.93%
e. Compressive Index (remoulded soil)	.48-.58

f. Standard Compaction

	Optimum Moisture Content	Maximum dry Density lbs./cu.ft.
Standard AASHO	22.0%	93
Modified AASHO	20.5%	102

g. Permeability- (determined for consolidated remoulded soil at range of consolidation pressures of 2 to 4 kg/cm^2) 1×10^{-9} cm/sec.

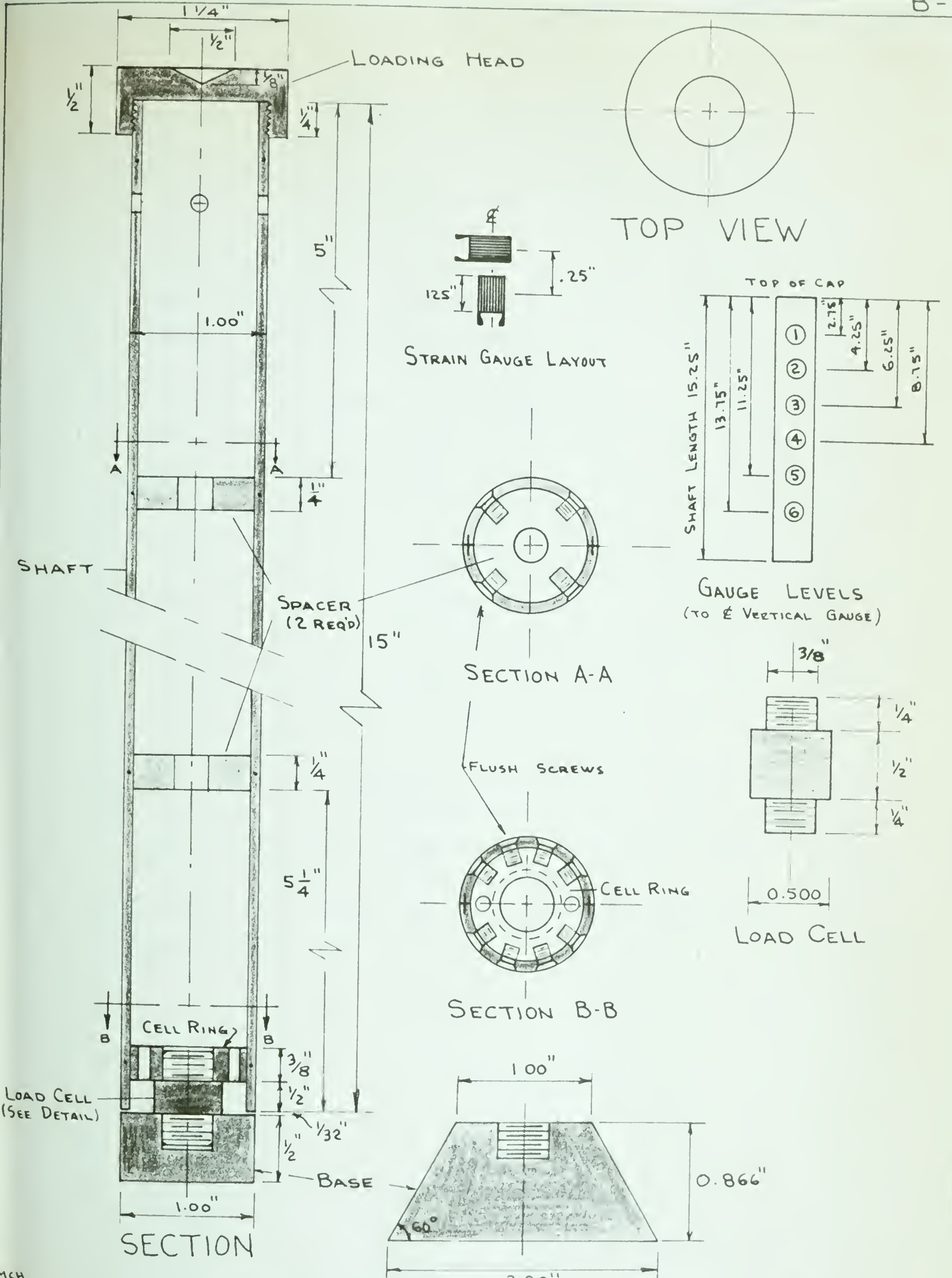
4. Mineralogical Composition:

	Percent of Clay Fraction
Montmorillonoids	30-40%
Illites	30-40%
Chlorite	20-30%

APPENDIX B

MODEL PILE DESIGN DETAILS

1. Drawing M 1a - Model Pile
2. Drawing M 1b - Instructions
3. Strain gauge and other technical data



MODEL PILE

DATE JUNE 26, 1963	SCALE NOT TO SCALE	DRAWING No. M-1a

FABRICATION INSTRUCTIONS

1. Note that drawing M-1a is not to scale.
2. Material - brass. If possible use hard red brass with yield strength of 40,000 psi.
3. Shaft to be of 1" O.D. brass tubing with 1/16" wall and split to provide two halves; pined both sides at 4 locations.
4. Only dimensions shown in decimals are critical.
5. All threads to be National Fine.
6. One side of spacers may be silver soldered in place.
7. Cell ring cannot be soldered in place. Screws incell ring to be as large as possible, root diameter about 1/8" .
8. Load cell to have smooth finish to take strain gauges. Load cell must bear on base and cell ring. Threaded section of load cell must not protrude above cell ring.
9. Two (2) bases required, one 1" diameter cylinder and one frustrum of cone with 1" top, 2" bottom and 60^{0} angle at base. Both bases tapped to take load cell. The 1" diameter of these bases is actually to be exactly same as O.D. of brass tubing.
10. Make cell ring, load cell and cylindrical base first.
11. The holes in the spacers, cell ring and at top of shaft are for the strain gauge wires and should be smoothed at edges to prevent cutting the wires.
11. Screws into spacers and cell ring to be flush with surface of shaft.

	MODEL PILE
DATE JUNE 26, 1963	INSTRUCTIONS	DRAWING NO. M-1b

MODEL PILE DETAILS

1. STRAIN GAUGE DATA

(a) Pile shaft and load cell

Gauges - Budd Metafilm, No. C 12-121, epoxy back, gauge factor 2.06 ± 1/2%, 120 ohms, gauge length 1/8 inch.

Glue - Tatnal contact glue GA - 1

Waterproofing - Budd GW - 1

(b) Brass tube

Gauges - Budd Metafilm, No. C 12-141, epoxy back, gauge factor 2.08 ± 1/2%, 120 ohms, gauge length 1/4 inch.

Glue - Tatnal contact glue GA-1, and household glue.

(c) Solid brass cylinder

Gauges - Baldwin-Lima, SR-4, Type A-5-S6, paper back, gauge factor 2.00 ± 1%, gauge length 1/2 inch.

Glue - Tatnal contact glue, GA-1

2. WIRE

(a) Pile shaft and load cell - thermocouple wire
- blue strand 27 ohms per thousand feet.
- red strand 770 ohms per thousand feet.

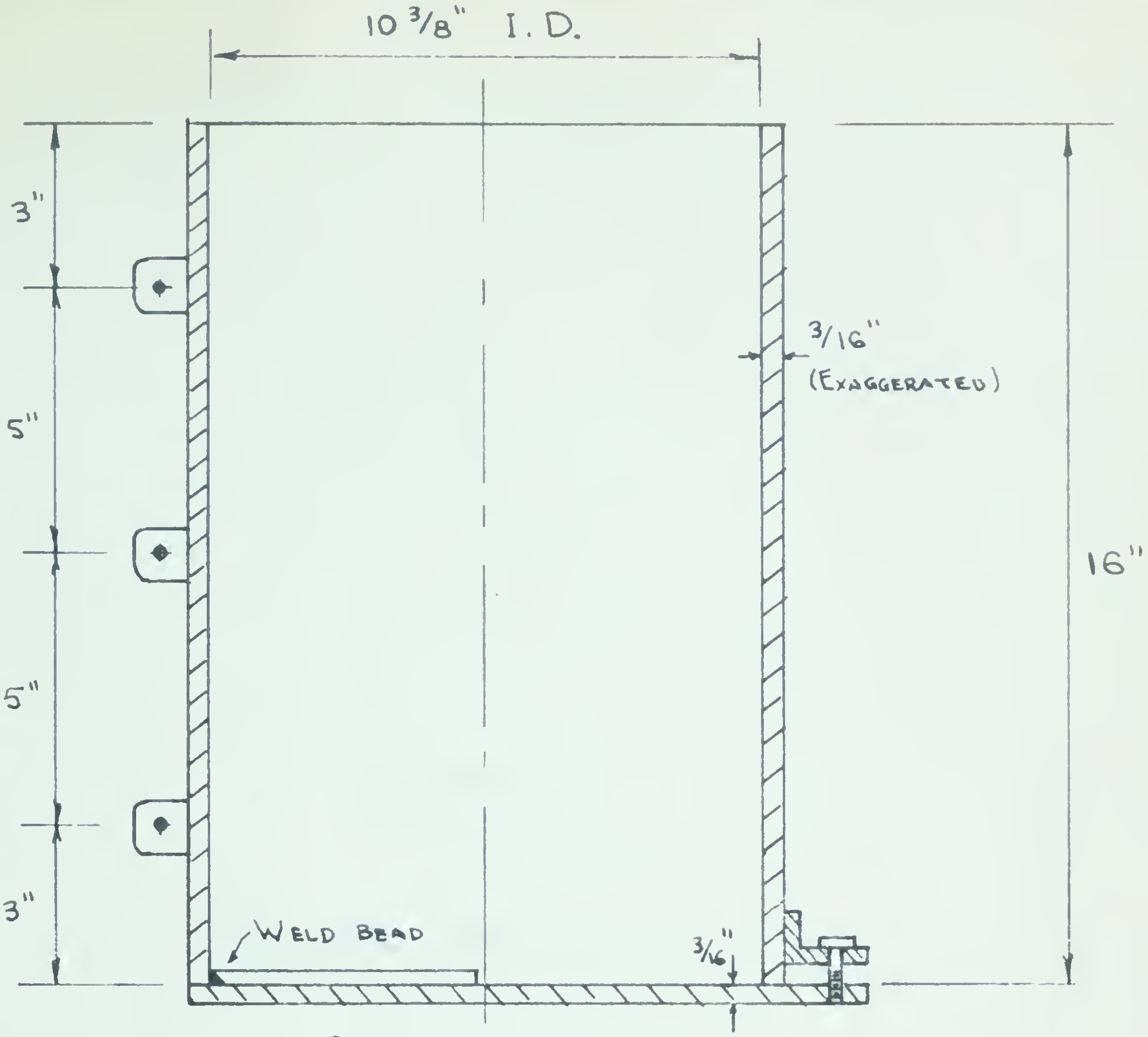
(b) Brass tube and solid brass cylinder
- Belden 22AWG 7x30 hook-up wire.

3. CONNECTORS (2)

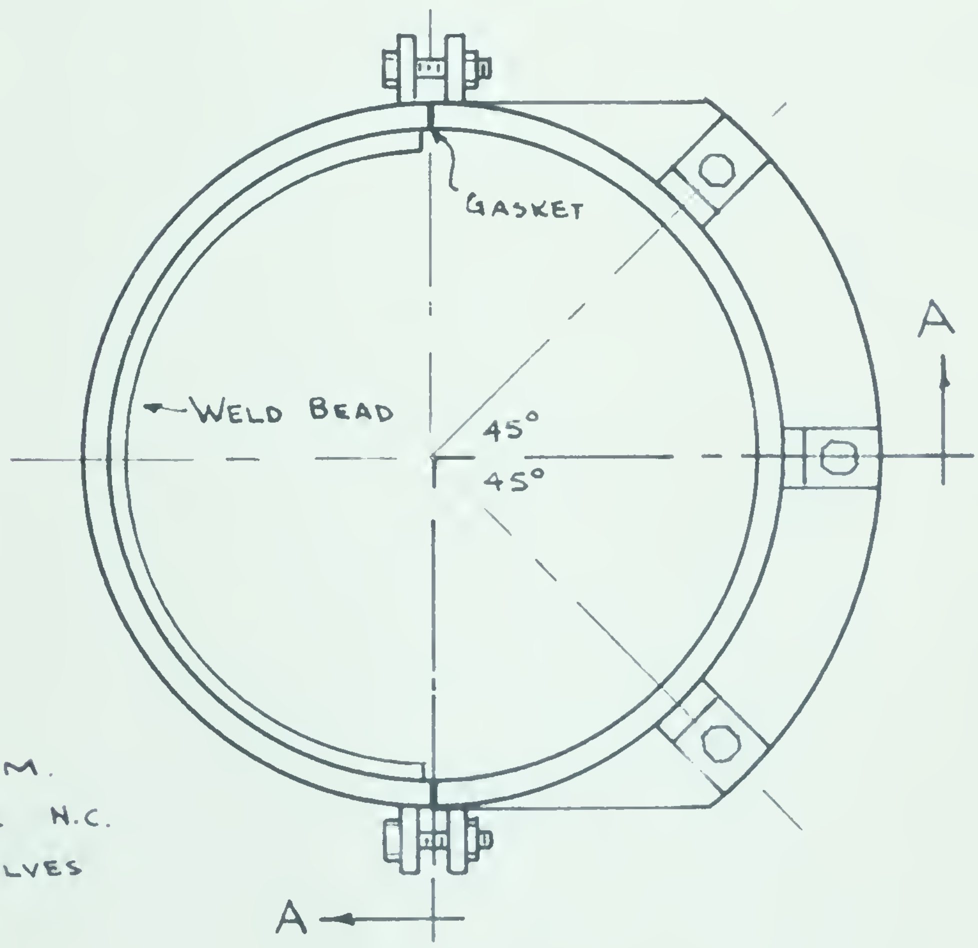
- plug-Cannon NK 27-22-C 3/4
- socket, chassis mounted-Cannon NK 27-31SL
- junction box- Hammond junction box chassis 1443-10, base 1432-10

APPENDIX C

DETAILS OF ALUMINIUM MOULD



SECTION A - A



PLAN

NOTES.

1. MATERIAL - ALUMINUM.
2. BOLTS & NUTS - STEEL N.C.
3. GASKET BETWEEN HALVES

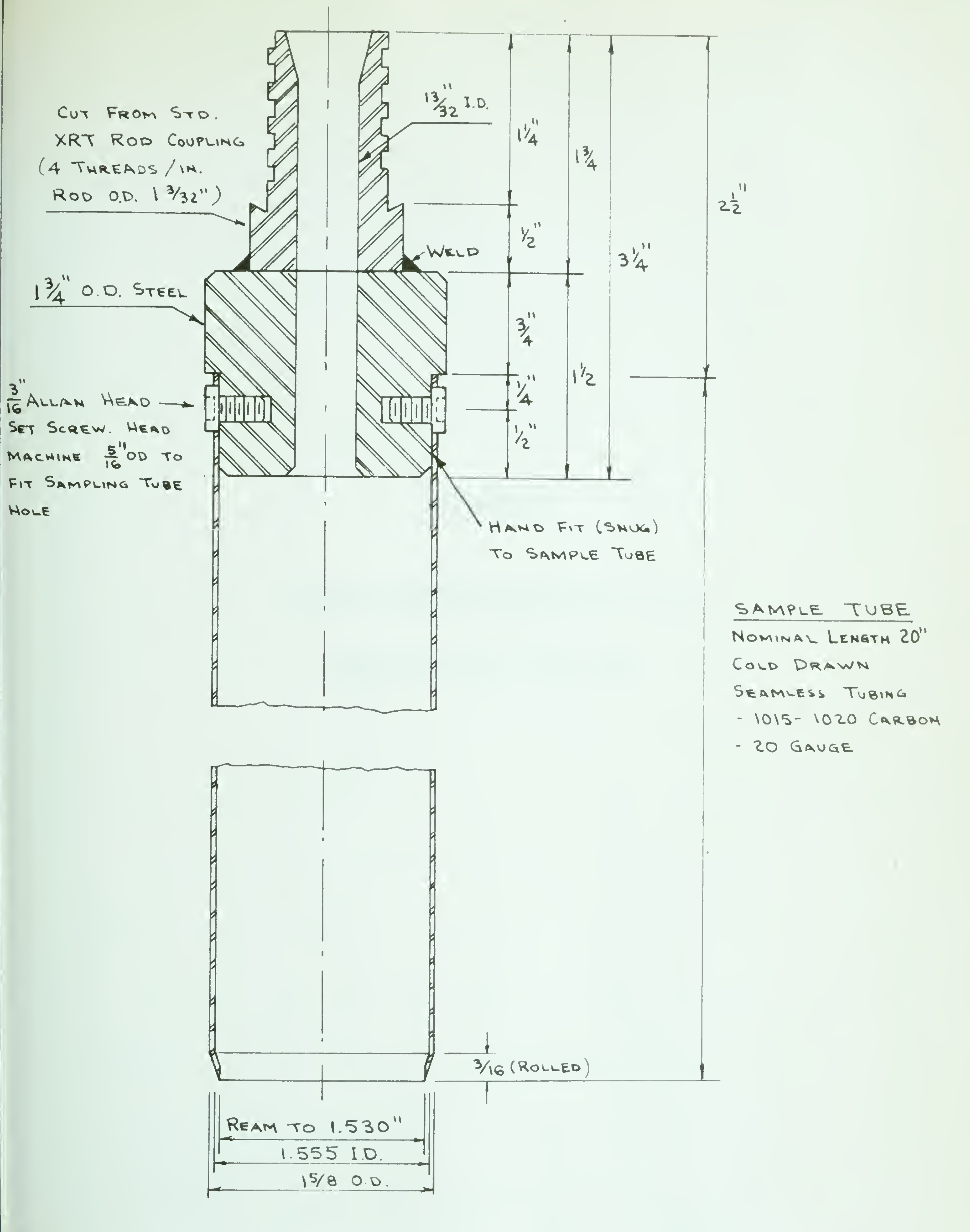
MCH

	MOULD

DATE JULY 2, 1963	SCALE - NOT TO SCALE	DRAWING No. M-2a

APPENDIX D

SAMPLING TUBE DESIGN



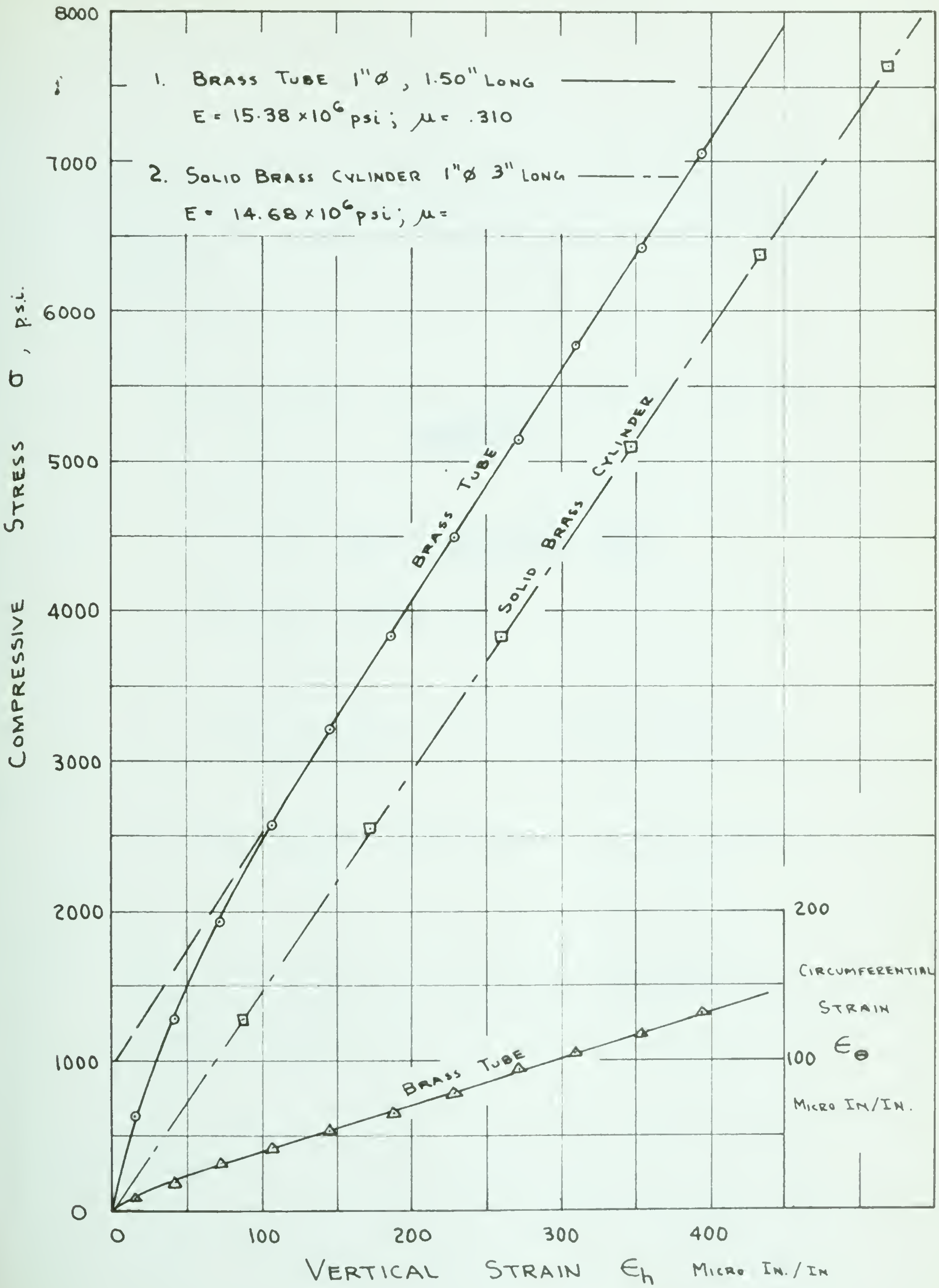
SAMPLER

DATE JULY 19, 1963	SCALE FULL SIZE	DRAWING NO. S-1

APPENDIX E

STRESS-STRAIN CURVES FOR 1" DIA.

BRASS TUBE AND CYLINDER



AXIAL COMPRESSION TESTS

APPENDIX F

PILE CALIBRATION DATA

PILE CALIBRATION DATA

OUTPUT FORMAT

The data from the strain gauge readings have been printed out in the form required by the IBM 1620 computer. The readings are repetitive and line-by-line the format is as follows:

Line No.	Output			
1	Test Number	Base Diameter	Pile Length	Shaft Length
eg.	101	2.00	16.120	16.120
2.	Pile Area (cm^2)	Thickness ratio (2t/b)		
eg.	1.070	0.1048		
3	Modulus of Elasticity of Pile (psix10^{-6})	Poisson's Ratio		
eg.	15.380	.310		
4	Number of Pile Loads to be analysed	Number of strain gauge readings		
eg.	7	6		
5	Proving Ring Load (kgms)	Penetration		
eg.	50.	0		

(Note: Penetration readings were not taken for any calibration tests and therefore no values were given on some data cards).

6 to 11 Strain gauge readings at each level of gauges (1-6 levels)(Inches x 10^{-4})

SVL -strain for vertical gauge left side of pile
SVR -strain for vertical gauge right side of pile
SHL -strain for horizontal gauge left side of pile
SHR -strain for horizontal gauge right side of pile

Gauge No.	SVL	SVR	SHL	SHR
eg. 1	60	30	-20	-5

(continued for all 6 levels of gauges, compressive strains taken as positive).

12 and on -Proving Ring Load and 6 gauge level readings repeated to end of test data.

CALIBRATION DATA TEST 101

```
101     2.00      16.120     16.120
 1.070     0.1048
15.38      0.31
 7    6
50.        0.
 1.       60.        30.        -20.       -5.
 2.       75.        25.        -20.       -5.
 3.       75.        55.        -25.       -5.
 4.       80.        30.        -20.       -5.
 5.       75.        10.        -20.        0.
 6.       70.       -50.        -20.       10.
100.       0.
 1.      110.        70.        -40.      -20.
 2.      120.        60.        -35.      -20.
 3.      125.        95.        -45.      -20.
 4.      130.        60.        -40.      -15.
 5.      125.        30.        -40.      -15.
 6.       90.      -225.        -30.       10.
150.       0.
 1.      140.       105.        -50.      -30.
 2.      155.       100.        -55.      -35.
 3.      165.       150.        -60.      -45.
 4.      160.       100.        -60.      -35.
 5.      165.        70.        -60.      -25.
 6.      110.      -300.        -20.       -5.
200.       0.
 1.      180.       150.        -70.      -55.
 2.      200.       150.        -70.      -55.
 3.      205.       195.        -80.      -65.
 4.      215.       145.        -70.      -55.
 5.      200.       115.        -80.      -50.
 6.      130.      -345.        -20.      -20.
300.       0.
 1.      280.       225.        -95.      -80.
 2.      285.       225.       -100.      -85.
 3.      295.       285.       -115.     -100.
 4.      285.       240.       -105.      -85.
 5.      275.       200.       -105.      -85.
 6.      185.      -340.        -25.      -50.
400.       0.
 1.      350.       305.       -125.     -120.
 2.      365.       305.       -135.     -115.
 3.      380.       375.       -150.     -140.
 4.      380.       330.       -135.     -115.
 5.      355.       290.       -135.     -125.
 6.      240.      -295.        -35.      -80.
500.       0.
 1.      440.       380.       -160.     -140.
 2.      455.       380.       -170.     -145.
 3.      470.       465.       -185.     -180.
 4.      450.       410.       -175.     -145.
 5.      425.       380.       -165.     -155.
 6.      305.      -250.        -45.     -115.
```

[illegible]

CALIBRATION DATA TEST 102

```
102    2.00    16.120    16.120
 1.070     0.1048
15.38      0.31
 7   6
 50.      0.
 1.       55.      50.      -5.      -10.
 2.       55.      45.      -5.      -10.
 3.       65.      50.      -10.     -10.
 4.       55.      50.      -10.     -10.
 5.       50.      45.      -10.     -10.
 6.       30.      80.       0.      -15.
 100.     0.
 1.       90.      90.      -20.     -25.
 2.       95.      90.      -20.     -25.
 3.       100.     90.      -20.     -35.
 4.       90.      95.      -20.     -30.
 5.       80.      90.      -20.     -30.
 6.        30.     100.      10.     -10.
 150.     0.
 1.       120.     130.     -40.     -40.
 2.       125.     130.     -40.     -40.
 3.       140.     135.     -40.     -60.
 4.       125.     145.     -40.     -50.
 5.       110.     135.     -40.     -55.
 6.        25.     130.      15.     -25.
 200.     0.
 1.       170.     165.     -50.     -60.
 2.       165.     175.     -55.     -60.
 3.       180.     175.     -55.     -75.
 4.       160.     195.     -55.     -65.
 5.       145.     175.     -55.     -75.
 6.        50.     160.      15.     -50.
 300.     0.
 1.       265.     230.     -90.     -90.
 2.       260.     240.     -90.     -85.
 3.       280.     250.     -90.     -110.
 4.       245.     275.     -85.     -95.
 5.       220.     265.     -85.     -110.
 6.       110.     240.      -5.     -90.
 400.     0.
 1.       360.     290.     -120.    -110.
 2.       355.     305.     -145.    -120.
 3.       375.     320.     -135.    -145.
 4.       335.     355.     -120.    -125.
 5.       300.     345.     -120.    -150.
 6.       175.     320.     -20.     -125.
 500.     0.
 1.       460.     350.     -160.    -140.
 2.       455.     360.     -180.    -140.
 3.       475.     380.     -180.    -175.
 4.       430.     425.     -150.    -150.
 5.       385.     420.     -150.    -180.
 6.       245.     410.      -40.    -160.
```

```
   CALIBRATION DATA  TEST 103
103     1.00      16.120    16.120
 1.070      0.1048
15.38       0.31
 9   6
 20.      0.
 1.       40.        10.        -10.       -10.
 2.       40.         5.        -10.         5.
 3.       40.         5.        -15.        10.
 4.       50.        -5.        -15.        10.
 5.       50.       -10.        -10.        10.
 6.       50.       -20.        -10.        10.
 50.      0.
 1.       85.        30.        -15.         5.
 2.       80.        20.        -20.         5.
 3.       85.        20.        -30.         5.
 4.       105.        5.        -30.         5.
 5.       100.      -10.        -30.        10.
 6.       80.       -50.        -20.        25.
 75.      0.
 1.       110.       40.        -25.        -5.
 2.       110.       40.        -30.       -5.
 3.       120.       20.        -35.         0.
 4.       130.       15.        -35.         0.
 5.       130.        0.        -40.        5.
 6.       100.      -50.        -20.        30.
 100.     0.
 1.       130.       60.        -30.       -10.
 2.       135.       50.        -40.       -10.
 3.       145.       45.        -45.        -5.
 4.       165.       35.        -50.        0.
 5.       165.        10.       -50.        5.
 6.       120.      -50.        -20.        25.
 150.     0.
 1.       185.       90.        -45.       -25.
 2.       190.       80.        -60.       -20.
 3.       205.       70.        -70.       -20.
 4.       225.       65.        -75.       -10.
 5.       220.       35.        -75.       -10.
 6.       165.      -35.        -40         20.
 200.     0.
 1.       230.       125.       -60.       -40.
 2.       235.       110.       -80.       -35.
 3.       255.       105.       -95.       -35.
 4.       275.       95.        -90.       -30.
 5.       270.       70.        -90.       -25.
 6.       200.       -5.        -50.         5.
 300.     0.
 1.       315.       190.       -95.       -60.
 2.       330.       190.       -120.      -60.
 3.       350.       175.       -130.      -70.
 4.       370.       175.       -125.      -50.
 5.       355.       140.       -120.      -55.
 6.       275.       70.        -70.       -30.
 400.     0.
 1.       405.       260.       -125.      -85.
```

2.	420.	260.	-155.	-90.
3.	450.	240.	-170.	-105.
4.	460.	250.	-160.	-80.
5.	440.	220.	-155.	-85.
6.	345.	165.	-90.	-60.
500.	0.			
1.	495.	330.	-155.	-100.
2.	510.	330.	-190.	-120.
3.	550.	315.	-215.	-140.
4.	550.	325.	-200.	-110.
5.	530.	300.	-190.	-125.
6.	420.	255.	-105.	-100.

CALIBRATION DATA TEST 104

```
104     2.00      16.120     16.120
  1.070      0.1048
 15.38       0.31
  4    6
  20.       0.
  1.        20.        30.         0.        -5.
  2.        15.        30.         0.        -10.
  3.        10.        35.         0.        -10.
  4.        15.        30.        -5.        -10.
  5.        10.        25.         0.        -10.
  6.        10.        30.        -5.         -5.
  50.       0.
  1.        50.        65.       -10.        -20.
  2.        40.        60.       -10.        -20.
  3.        40.        65.       -10.        -20.
  4.        45.        60.       -10.        -15.
  5.        35.        55.       -10.        -20.
  6.        30.        55.         0.        -10.
  100.      0.
  1.        80.        110.      -20.        -40.
  2.        70.        105.      -20.        -40.
  3.        70.        105.      -25.        -40.
  4.        75.        110.      -20.        -35.
  5.        70.        100.      -20.        -35.
  6.        45.        85.        15.        -20.
  200.      0.
  1.        160.       200.      -45.        -70.
  2.        140.       200.      -50.        -70.
  3.        150.       200.      -55.        -80.
  4.        155.       215.      -55.        -65.
  5.        145.       185.      -50.        -75.
  6.        90.        155.       15.         -50.
```

CALIBRATION DATA TEST 201

```
201     1.        15.5
  1.070      0.1048
 15.380      0.310
  6   6
  100.
  1.       80.        60.        -50.       -50.
  2.       70.        60.        -50.       -40.
  3.       80.        60.        -50.       -50.
  4.       70.        70.        -70.       -70.
  5.       50.        50.        -50.       -60.
  6.      -60.       -30.         60.       -20.
  200.
  1.      170.       120.        -80.       -80.
  2.      150.       130.        -90.       -80.
  3.      170.       130.        -80.       -80.
  4.      150.       150.       -120.      -120.
  5.      120.       120.        -70.      -100.
  6.      -20.        40.         70.       -60.
  300.
  1.      260.       180.       -120.      -110.
  2.      240.       190.       -130.      -110.
  3.      250.       200.       -130.      -120.
  4.      220.       230.       -130.      -160.
  5.      180.       200.       -110.      -140.
  6.       10.       110.         60.      -100.
  400.
  1.      350.       230.       -160.      -130.
  2.      330.       260.       -170.      -130.
  3.      350.       260.       -170.      -160.
  4.      310.       300.       -190.      -190.
  5.      230.       280.       -140.      -200.
  6.       70.       200.         50.      -150.
  500.
  1.      450.       290.       -190.      -150.
  2.      430.       310.       -210.      -150.
  3.      450.       320.       -200.      -180.
  4.      390.       380.       -220.      -220.
  5.      330.       370.       -170.      -260.
  6.      120.       300.         40.      -180.
  600.
  1.      570.       320.       -240.      -170.
  2.      560.       350.       -260.      -170.
  3.      470.       360.       -240.      -200.
  4.      490.       450.       -250.      -240.
  5.      420.       440.       -250.      -210.
  6.      170.       390.         30.      -110.
```

APPENDIX G

PILE TEST DATA

PILE TEST DATA

OUTPUT FORMAT

The readings from the strain gauges, penetration dial, proving ring and load cell have been printed out in the form required by the IBM 1620 Computer. The readings are repetitive and, line-by-line, the format is as follows:

Line	Output		
1	Test Number, Base Diameter, Base depth and Shaft Length, Inches; Maximum load at failure (kgms)		
eg.	111 (A-1a) 2.0 10.88 9.85 245		
2	Pile area (cm^2)	Thickness ratio (2t/b)	
eg.	1.070	0.1048	
3	Modulus of elasticity of pile ($psi \times 10^{-6}$)	Poisson's Ratio	
eg.	15.380	.310	
4.	Portion of shaft length for which gauge level gives average vertical strain (inches); 6 levels.		
eg.	3.5 1.75 2.25 2.5 2.5 2.06		
5.	Number of Pile loads	Number of gauge levels	
eg.	22	6	
6	Proving Ring Load, kgms.	Penetration ($inches \times 10^{-4}$)	Base Load, from load cell, kg/cm^2.
	20	7	7

7 to 12	Gauge depth (GDEP) below soil (inches); strain reading for each gauge at that gauge depth (inches x 10^{-4}).

SVL -strain for vertical gauge, left side of pile
SVR -strain for vertical gauge, right side of pile
SHL -strain for horizontal gauge, left side of pile
SHR -strain for horizontal gauge, right side of pile.

	GDEP	SVL	SVR	SHL	SHR
eg.	-2.44	20	15	-20	-15

(the minus gauge depth indicates a level above the soil; compressive strains are taken as positive).

13 and on — Proving ring loads etc. repeated to end of test data.

```
   PILE TEST DATA   TEST 111
111    2.      10.88      9.85    245.
 1.070     0.1048
15.380     0.310
 3.5     1.75      2.25      2.5       2.5       2.06
22    6
 20.     7.        7.
 -2.44   20.       15.       -20.      -5.
 -0.94   20.       20.       -5.       -5.
 1.06    20.       15.       -5.       -5.
 3.56    20.       15.       -5.       0.
 6.06    10.       15.       5.        -5.
 8.56    10.       5.        0.        0.
 40.     12.       14.
 -2.44   40.       35.       -10.      -10.
 -0.94   40.       35.       -10.      -5.
 1.06    35.       40.       -10.      -10.
 3.56    35.       30.       -10.      -10.
 6.06    25.       25.       -5.       -10.
 8.56    15.       15.       0.        0.
 54.2    17.       18.
 -2.44   50.       45.       -10.      -15.
 -0.94   55.       50.       -15.      -15.
 1.06    50.       55.       -15.      -15.
 3.56    50.       45.       -10.      -10.
 6.06    40.       35.       -5.       -10.
 8.56    25.       20.       0.        0.
 80.     26.       34.
 -2.44   70.       75.       -20.      -20.
 -0.94   80.       75.       -20.      -15.
 1.06    75.       85.       -20.      -20.
 3.56    70.       70.       -15.      -15.
 6.06    60.       55.       -15.      -20.
 8.56    40.       40.       -5.       -5.
 100.    36.       44.
 -2.44   90.       95.       -20.      -25.
 -0.94   95.       90.       -20.      -25.
 1.06    90.       105.      -20.      -30.
 3.56    90.       85.       -20.      -20.
 6.06    80.       75.       -20.      -20.
 8.56    55.       55.       -10.      -5.
 120.    59.       63.
 -2.44   105.      115.      -25.      -30.
 -0.94   110.      110.      -30.      -30.
 1.06    105.      125.      -25.      -30.
 3.56    110.      105.      -25.      -25.
 6.06    100.      90.       -25.      -30.
 8.56    75.       70.       -20.      -20.
 140.    111.      84.
 -2.44   120.      130.      -30.      -35.
 -0.94   125.      125.      -40.      -35.
 1.06    120.      135.      -35.      -40.
 3.56    125.      125.      -30.      -30.
 6.06    115.      105.      -35.      -35.
 8.56    80.       85.       -20.      -15.
 160.    189.      105.
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-2.44	140.	145.	-40.	-45.
-0.94	140.	140.	-45.	-45.
1.06	140.	155.	-40.	-50.
3.56	145.	145.	-40.	-40.
6.06	130.	125.	-35.	-40.
8.56	85.	95.	-15.	-30.
180.	326.	130.		
-2.44	155.	175.	-40.	-50.
-0.94	160.	170.	-50.	-45.
1.06	155.	175.	-45.	-55.
3.56	155.	165.	-45.	-45.
6.06	145.	155.	-35.	-45.
8.56	90.	100.	-10.	-30.
200.	545.	155.		
-2.44	170.	185.	-50.	-55.
-0.94	175.	190.	-55.	-55.
1.06	175.	195.	-50.	-65.
3.56	170.	185.	-50.	-50.
6.06	160.	155.	-35.	-45.
8.56	95.	95.	-10.	-30.
218.	903.	176.		
-2.44	180.	195.	-50.	-55.
-0.94	185.	200.	-60.	-60.
1.06	190.	210.	-55.	-70.
3.56	180.	215.	-45.	-55.
6.06	180.	190.	-35.	-40.
8.56	110.	85.	-15.	-30.
225.	1295.	186.		
-2.44	185.	205.	-55.	-60.
1.06	195.	215.	-60.	-75.
-0.94	195.	205.	-60.	-65.
3.56	195.	220.	-60.	-60.
6.06	190.	195.	-35.	-40.
8.56	120.	80.	-20.	-30.
235.	1888.	203.		
-2.44	190.	215.	-60.	-65.
-0.94	195.	210.	-65.	-65.
1.06	200.	230.	-65.	-85.
3.56	195.	235.	-65.	-65.
6.06	205.	195.	-40.	-40.
8.56	130.	85.	-20.	-35.
245.	3140.	214.		
-2.44	190.	215.	-65.	-65.
-0.94	210.	215.	-70.	-65.
1.06	215.	230.	-70.	-90.
3.56	210.	235.	-70.	-65.
6.06	220.	195.	-45.	-30.
8.56	145.	80.	-20.	-30.
160.	3244.	186.		
-2.44	130.	140.	-45.	-50.
-0.94	140.	140.	-50.	-45.
1.06	140.	160.	-50.	-70.
3.56	150.	175.	-45.	-50.
6.06	180.	145.	-35.	-20.
8.56	115.	50.	-20.	-15.
100.	3220.	165.		

-2.44	80.	85.	-30.	-30.
-0.94	90.	80.	-35.	-25.
1.06	90.	105.	-35.	-50.
3.56	110.	125.	-40.	-40.
6.06	140.	115.	-25.	-10.
8.56	95.	35.	-10.	-20.
0.	2826.	57.		
-2.44	-10.	15.	-10.	0.
-0.94	-5.	-10.	-10.	-10.
1.06	-5.	20.	-10.	-20.
3.56	30.	25.	-25.	-25.
6.06	65.	10.	-15.	10.
8.56	110.	65.	10.	-10.
0.	2754.	58.8		
-2.44	-5.	-5.	-10.	-10.
-0.94	0.	-10.	0.	-5.
1.06	5.	15.	-10.	-20.
3.56	35.	25.	-25.	-20.
6.06	70.	15.	-15.	10.
8.56	15.	-10.	15.	-10.
0.	2747.	53.		
-2.44	-10.	-10.	-10.	-10.
-0.94	-5.	-10.	-10.	-5.
1.06	0.	5.	-15.	-20.
3.56	30.	20.	-20.	-20.
6.06	70.	5.	-15.	10.
8.56	15.	-20.	20.	-5.
0.	2745.	42.		
-2.44	-5.	-5.	-10.	-10.
-0.94	0.	-10.	-5.	0.
1.06	5.	-5.	-10.	-15.
3.56	30.	15.	-15.	-15.
6.06	65.	-5.	-15.	10.
8.56	10.	10.	15.	0.
0.	2748.	35.		
-2.44	0.	5.	0.	0.
-0.94	0.	0.	0.	5.
1.06	5.	-20.	-5.	-15.
3.56	30.	20.	-15.	-10.
6.06	65.	-15.	-15.	15.
8.56	10.	25.	10.	-5.
0.	2749.	31.		
-2.44	0.	5.	0.	0.
-0.94	5.	0.	0.	5.
1.06	5.	-25	-5.	-15.
3.56	25.	15.	-10.	-10.
6.06	60.	-65.	-15.	15.
8.56	10.	-115.	10.	-5.

PILE TEST DATA TEST 112

112	2.	11.23	10.20	282.	
1.070	0.1048				
15.380	0.310				
3.5	1.75	2.25	2.5	2.5	2.06
13	6				
40.	12.5	9.7			
-2.09	35.	35.	-10.	-10.	
-0.59	30.	35.	-10.	-10.	
1.41	25.	35.	-15.	-15.	
3.91	25.	30.	-10.	-10.	
6.41	20.	20.	-5.	-5.	
8.91	10.	10.	0.	0.	
80.	22.5	21.1			
-2.09	70.	80.	-20.	-20.	
-0.59	65.	70.	-20.	-20.	
1.41	60.	70.	-25.	-25.	
3.91	55.	55.	-15.	-15.	
6.41	40.	50.	-10.	-10.	
8.91	25.	25.	-5.	0.	
100.	28.	23.9			
-2.09	85.	105.	-20.	-30.	
-0.59	85.	90.	-25.	-25.	
1.41	80.	90.	-30.	-20.	
3.91	70.	85.	-20.	-20.	
6.41	50.	55.	-15.	-15.	
8.91	30.	30.	-5.	-5.	
140.	40.	37.9			
-2.09	115.	130.	-30.	-40.	
-0.59	115.	130.	-35.	-35.	
1.41	110.	120.	-40.	-45.	
3.91	95.	100.	-30.	-30.	
6.41	70.	70.	-15.	-20.	
8.91	40.	45.	-10.	-10.	
180.	66.	61.6			
-2.09	150.	170.	-40.	-45.	
-0.59	140.	165.	-45.	-45.	
1.41	145.	155.	-50.	-55.	
3.91	125.	140.	-35.	-40.	
6.41	90.	100.	-15.	-25.	
8.91	65.	70.	-15.	-15.	
200.	130.	98.			
-2.09	170.	190.	-40.	-50.	
-0.59	170.	185.	-50.	-50.	
1.41	165.	175.	-60.	-65.	
3.91	145.	160.	-40.	-45.	
6.41	115.	125.	-10.	-25.	
8.91	90.	90.	-20.	-15.	
225.	253.	120.			
-2.09	190.	210.	-45.	-55.	
-0.59	190.	210.	-55.	-55.	
1.41	190.	200.	-65.	-75.	
3.91	165.	195.	-50.	-55.	
6.41	135.	145.	0.	-25.	
8.91	100.	100.	-25.	-15.	
282.	1355.	188.			

-2.09	230.	260.	-65.	-70.
-0.59	235.	235.	-70.	-70.
1.41	235.	245.	-85.	-105.
3.91	205.	250.	-55.	-70.
6.41	200.	205.	0.	-10.
8.91	165.	130.	-15.	-10.
180.	1806.	150.		
-2.09	145.	170.	-40.	-45.
-0.59	150.	165.	-45.	-40.
1.41	145.	155.	-60.	-75.
3.91	140.	170.	-40.	-55.
6.41	150.	150.	10.	0.
8.91	110.	80.	-10.	-10.
100.	1765.	111.		
-2.09	85.	100.	-20.	-25.
-0.59	80.	85.	-25.	-20.
1.41	85.	95.	-40.	-45.
3.91	85.	115.	-30.	-40.
6.41	110.	110.	10.	5.
8.91	80.	50.	0.	0.
0.	1370.	2.5		
-2.09	-5.	-5.	-5.	-5.
-0.59	-10.	-10.	0.	0.
1.41	-5.	-5.	-20.	-15.
3.91	5.	10.	-10.	-15.
6.41	20.	10.	10.	15.
8.91	0.	-25.	15.	10.
0.	1270.	5.		
-2.09	-5.	0.	0.	0.
-0.59	-5.	-5.	0.	5.
1.41	-5.	-5.	-20.	-25.
3.91	10.	10.	-10.	-15.
6.41	25.	15.	10.	15.
8.91	0.	-10.	15.	10.
0.	1270.	-5.		
-2.09	-5.	0.	5.	0.
-0.59	-5.	-5.	5.	10.
1.41	-5.	-5.	-15	-5.
3.91	15.	20.	-10.	-15.
6.41	30.	-10.	5.	15.
8.91	0.	-35.	15.	5.

PILE TEST DATA TEST 121

121	1.	11.92	11.12	200.	
1.070	0.1048				
15.380	0.310				
3.5	1.75	2.25	2.5	2.5	2.06
12	6				
20.	3.	0.			
-1.25	30.	25.	0.	-5.	
0.25	30.	35.	0.	-5.	
2.25	25.	15.	-5.	-5.	
4.75	10.	0.	-5.	0.	
7.25	0.	5.	-5.	0.	
9.75	0.	5.	0.	5.	
40.	5.2	1.			
-1.25	40.	40.	-5.	-5.	
0.25	60.	40.	-5.	-5.	
2.25	30.	30.	-5.	-15.	
4.75	20.	15.	-5.	-5.	
7.25	10.	10.	-5.	-5.	
9.75	5.	5.	0.	0.	
60.	6.8	2.			
-1.25	60.	55.	-10.	-10.	
0.25	60.	60.	-10.	-10.	
2.25	60.	40.	-15.	-15.	
4.75	30.	25.	-10.	-10.	
7.25	10.	20.	-10.	-5.	
9.75	5.	10.	-5.	-5.	
80.	7.	2.			
-1.25	135.	125.	-20.	-20.	
0.25	145.	140.	-15.	-15.	
2.25	110.	50.	-20.	-25.	
4.75	40.	35.	-10.	-10.	
7.25	15.	20.	-10.	-5.	
9.75	5.	5.	-10.	-5.	
100.	7.2	3.5			
-1.25	155.	145.	-30.	-25.	
0.25	155.	160.	-20.	-15.	
2.25	125.	70.	-25.	-30.	
4.75	50.	40.	-15.	-15.	
7.25	20.	20.	-10.	-10.	
9.75	5.	10.	-10.	-5.	
140.	10.	3.5			
-1.25	180.	180.	-35.	-35.	
0.25	180.	195.	-20.	-20.	
2.25	150.	80.	-35.	-40.	
4.75	65.	60.	-20.	-20.	
7.25	25.	30.	-10.	-10.	
9.75	5.	10.	-10.	-5.	
180.	17.8	5.			
-1.25	215.	215.	-50.	-50.	
0.25	215.	240.	-30.	-30.	
2.25	175.	115.	-40.	-55.	
4.75	85.	85.	-30.	-30.	
7.25	35.	40.	-15.	-10.	
9.75	5.	10.	-15.	-5.	
200.	29.	5.			

-1.25	235.	235.	-50.	-50.
0.25	230.	260.	-30.	-30.
2.25	190.	140.	-45.	-55.
4.75	95.	100.	-35.	-35.
7.25	40.	40.	-15.	-10.
9.75	10.	10.	-20.	-15.
100.	232.	-1.		
-1.25	155.	165.	-20.	-20.
0.25	155.	175.	5.	0.
2.25	120.	70.	-15.	-10.
4.75	45.	55.	-15.	-20.
7.25	0.	20.	0.	10.
9.75	0.	-5.	-30.	-5.
0.	218.	-5.		
-1.25	70.	60.	5.	5.
0.25	80.	35.	20.	10.
2.25	50.	10.	10.	15.
4.75	-5.	5.	-5.	-10.
7.25	-30.	-15.	5.	10.
9.75	-10.	-20.	-30.	-5.
0.	216.	-3.5		
-1.25	75.	65.	0.	0.
0.25	80.	85.	20.	15.
2.25	55.	0.	10.	10.
4.75	0.	5.	-5.	-10.
7.25	-25.	-10.	0.	10.
9.75	-5.	-20.	-35.	-10.
0.	217.	0.		
-1.25	90.	75.	10.	10.
0.25	95.	100.	15.	10.
2.25	60.	10.	5.	5.
4.75	5.	20.	-5.	-15.
7.25	-15.	-5.	-5.	10.
9.75	5.	-10.	-40.	-15.

PILE TEST DATA TEST 122

122	1.	11.97	11.17	220.	
1.070	0.1048				
15.380	0.310				
3.5	1.75	2.25	2.5	2.5	2.06
11	6				
40.	6.	0.			
-0.95	30.	40.	-15.	-5.	
0.55	30.	40.	-5.	-5.	
2.55	25.	25.	-10.	-10.	
5.05	15.	20.	-5.	-5.	
7.55	15.	15.	-5.	-10.	
10.05	5.	10.	0.	0.	
80.	8.8	1.3			
-0.95	70.	70.	-25.	-15.	
0.55	60.	70.	-15.	-15.	
2.55	50.	55.	-15.	-20.	
5.05	35.	40.	-10.	-25.	
7.55	25.	25.	-5.	-10.	
10.05	5.	20.	-5.	-10.	
100.	10.8	0.			
-0.95	80.	90.	-35.	-15.	
0.55	75.	85.	-15.	-15.	
2.55	65.	65.	-20.	-25.	
5.05	45.	50.	-10.	-15.	
7.55	30.	35.	-10.	-5.	
10.05	5.	20.	-5.	-5.	
140.	15.2	5.			
-0.95	110.	130.	-40.	-30.	
0.55	105.	130.	-20.	-25.	
2.55	95.	95.	-30.	-35.	
5.05	65.	60.	-20.	-20.	
7.55	35.	40.	-10.	-15.	
10.05	5.	20.	-10.	-5.	
161.	93.	18.			
-0.95	145.	160.	-45.	-35.	
0.55	135.	160.	-15.	-20.	
2.55	120.	120.	-30.	-30.	
5.05	90.	90.	-25.	-25.	
7.55	55.	65.	-5.	0.	
10.05	35.	30.	-20.	-10.	
220.	853.	92.			
-0.95	200.	200.	-55.	-45.	
0.55	175.	210.	-5.	-25.	
2.55	180.	160.	-35.	-40.	
5.05	155.	140.	-35.	-35.	
7.55	135.	125.	0.	-10.	
10.05	55.	50.	-20.	10.	
120.	1103.	80.			
-0.95	100.	125.	-30.	-40.	
0.55	95.	130.	20.	0.	
2.55	100.	100.	-10.	-25.	
5.05	95.	90.	-25.	-25.	
7.55	85.	85.	5.	20.	
10.05	15.	25.	10.	-10.	
40.	1093.	62.			

-0.95	25.	55.	-5.	-10.
0.55	30.	55.	35.	15.
2.55	40.	50.	5.	0.
5.05	45.	50.	-15.	-15.
7.55	50.	55.	15.	25.
10.05	0.	0.	-5.	10.
0.	1086.	52.		
-0.95	-5.	10.	0.	0.
0.55	5.	10.	40.	25.
2.55	10.	20.	0.	5.
5.05	15.	30.	-5.	-10.
7.55	35.	35.	20.	30.
10.05	-10.	-10.	0.	5.
0.	1076.	43.		
-0.95	0.	10.	0.	5.
0.55	0.	10.	25.	10.
2.55	10.	30.	5.	-5.
5.05	15.	25.	-10.	-15.
7.55	25.	35.	5.	20.
10.05	-20.	30.	-10.	-5.
0.	1075.	41.		
-0.95	0.	10.	0.	0.
0.55	0.	10.	25.	5.
2.55	15.	40.	0.	-5.
5.05	5.	40.	-5.	-20.
7.55	25.	35.	5.	20.
10.05	-15.	20.	-5.	-5.

PILE TEST DATA TEST 130

130	2.	11.9	10.75	543.	
1.070	0.1048				
15.380	0.310				
3.5	1.75	2.25	2.5	2.5	2.06
16 6					
20.	2.5	5.			
-1.42	20.	10.	-5.	-5.	
0.08	20.	10.	-10.	-5.	
2.08	15.	25.	-5.	-5.	
4.58	10.	10.	-5.	-5.	
7.08	10.	10.	0.	-5.	
9.58	5.	5.	-5.	-5.	
40.	4.2	7.5			
-1.42	40.	30.	-10.	-10.	
0.08	40.	20.	-10.	-10.	
2.08	30.	45.	-10.	-10.	
4.58	20.	25.	-10.	-10.	
7.08	15.	15.	-5.	0.	
9.58	10.	10.	-5.	-5.	
60.	6.3	9.			
-1.42	60.	45.	-20.	-10.	
0.08	60.	40.	-20.	-10.	
2.08	45.	60.	-15.	-10.	
4.58	35.	35.	-10.	-10.	
7.08	20.	20.	-5.	0.	
9.58	10.	10.	-5.	-10.	
80.	10.	12.			
-1.42	70.	60.	-15.	-15.	
0.08	80.	50.	-25.	-10.	
2.08	60.	75.	-25.	-15.	
4.58	50.	45.	-15.	-15.	
7.08	30.	30.	-5.	-5.	
9.58	20.	20.	-5.	-5.	
100.	15.9	11.			
-1.42	90.	80.	-25.	-25.	
0.08	105.	65.	-30.	-20.	
2.08	70.	85.	-30.	-20.	
4.58	60.	55.	-20.	-20.	
7.08	40.	30.	-10.	-10.	
9.58	20.	20.	-10.	-10.	
120.	21.7	11.			
-1.42	110.	95.	-30.	-20.	
0.08	120.	80.	-30.	-20.	
2.08	100.	105.	-35.	-20.	
4.58	70.	65.	-10.	-20.	
7.08	45.	40.	-10.	-10.	
9.58	20.	30.	-5.	-10.	
140.	27.5	13.			
-1.42	120.	110.	-35.	-35.	
0.08	140.	90.	-40.	-20.	
2.08	110.	115.	-40.	-30.	
4.58	90.	80.	-25.	-25.	
7.08	50.	50.	-15.	-10.	
9.58	30.	30.	-10.	-10.	
160.	36.8	14.			

-1.42	140.	130.	-30.	-40.
0.08	160.	100.	-40.	-25.
2.08	130.	125.	-45.	-30.
4.58	100.	95.	-30.	-30.
7.08	70.	65.	-15.	-15.
9.58	30.	40.	-15.	-10.
180.	47.1	18.		
-1.42	160.	150.	-45.	-30.
0.08	180.	130.	-45.	-25.
2.08	150.	145.	-55.	-40.
4.58	120.	115.	-35.	-35.
7.08	85.	80.	-20.	-20.
9.58	40.	50.	-15.	-20.
200.	65.4	42.5		
-1.42	180.	165.	-50.	-30.
0.08	200.	150.	-50.	-30.
2.08	175.	160.	-65.	-40.
4.58	140.	135.	-35.	-40.
7.08	110.	100.	-25.	-20.
9.58	60.	70.	-15.	-25.
543.	5875.	470.		
-1.42	540.	370.	-145.	-100.
0.08	555.	360.	-90.	-100.
2.08	565.	355.	-135.	-140.
4.58	470.	430.	-135.	-135.
7.08	380.	430.	-135.	-170.
9.58	260.	410.	-80.	-80.
300.	6105.	377.		
-1.42	300.	210.	-85.	-40.
0.08	305.	200.	-50.	-60.
2.08	350.	205.	-150.	-190.
4.58	310.	185.	-130.	-100.
7.08	255.	310.	-110.	-135.
9.58	170.	310.	-55.	-155.
0.	5670.	62.		
-1.42	-10.	-10.	-15.	-5.
0.08	-5.	-10.	25.	10.
2.08	20.	5.	-35.	-15.
4.58	25.	20.	-30.	-35.
7.08	-15.	50.	-15.	-50.
9.58	-95.	50.	15.	-70.
0.	5619.	97.		
-1.42	-10.	-10.	-5.	0.
0.08	10.	-5.	25.	10.
2.08	25.	25.	-25.	-5.
4.58	35.	40.	-20.	-30.
7.08	-15.	60.	-15.	-40.
9.58	-75.	80.	25.	-55.
0.	5613.	97.		
-1.42	-20.	-10.	-5.	-5.
0.08	10.	-5.	25.	10.
2.08	25.	15.	-25.	-5.
4.58	35.	40.	-20.	-30.
7.08	-20.	35.	-15.	-40.
9.58	-70.	80.	15.	-70.
0.	5613.	98.		

-1.42	-10.	-10.	-5.	0.
0.08	10.	0.	20.	10.
2.08	20.	5.	-25.	-5.
4.58	30.	40.	-20.	-35.
7.08	-20.	35.	-15.	-40.
9.58	-70.	85.	15.	-70.

PILE TEST DATA TEST 140

140 1.	12.35	11.55	114.8		
1.070	0.1048				
15.380	0.310				
3.5	1.75	2.25	2.5	2.5	2.06
10 6					
20.	6.	4.			
-0.59	15.	20.	0.	-5.	
0.91	20.	20.	-5.	-5.	
2.91	15.	20.	-5.	0.	
5.41	10.	10.	0.	0.	
7.91	5.	10.	0.	0.	
10.41	0.	5.	0.	-10.	
40.	11.4	4.			
-0.59	30.	35.	-5.	-5.	
0.91	43.	35.	-10.	-5.	
2.91	30.	50.	-5.	-10.	
5.41	20.	30.	0.	0.	
7.91	10.	15.	0.	0.	
10.41	10.	10.	10.	0.	
60.	17.5	5.			
-0.59	45.	55.	-5.	-10.	
0.91	50.	55.	-10.	-10.	
2.91	50.	70.	-15.	-20.	
5.41	40.	45.	-10.	-10.	
7.91	20.	25.	0.	-5.	
10.41	10.	15.	0.	-5.	
80.	25.	6.			
-0.59	50.	75.	-10.	-15.	
0.91	70.	75.	-10.	-15.	
2.91	60.	85.	-15.	-15.	
5.41	55.	60.	-10.	-10.	
7.91	35.	40.	-5.	-10.	
10.41	20.	25.	0.	0.	
100.	37.4	9.			
-0.59	70.	100.	-15.	-25.	
0.91	80.	95.	-10.	-15.	
2.91	80.	100.	-25.	-30.	
5.41	70.	80.	-15.	-20.	
7.91	50.	50.	-10.	-10.	
10.41	30.	30.	0.	-10.	
114.8	1043.	57.6			
-0.59	100.	120.	-15.	-20.	
0.91	100.	120.	15.	15.	
2.91	100.	125.	-20.	-35.	
5.41	110.	110.	-35.	-35.	
7.91	90.	90.	-25.	-20.	
10.41	70.	75.	-15.	-20.	
60.	1507.	45.			
-0.59	40.	80.	25.	-5.	
0.91	55.	75.	25.	25.	
2.91	5.	90.	-10.	-25.	
5.41	70.	80.	-20.	-25.	
7.91	65.	70.	-20.	-10.	
10.41	60.	55.	-20.	-20.	
0.	1480.	36.6			

-0.59	5.	15.	-10.	-5.
0.91	20.	10.	30.	45.
2.91	20.	40.	-5.	-5.
5.41	45.	45.	-10.	-10.
7.91	40.	45.	-10.	-5.
10.41	50.	45.	-10.	-20.
0.	1473.	25.3		
-0.59	0.	10.	-10.	0.
0.91	15.	10.	20.	35.
2.91	10.	0.	-5.	-5.
5.41	35.	30.	-10.	-10.
7.91	30.	30.	-10.	-5.
10.41	40.	55.	-10.	-20.
0.	1472.	17.		
-0.59	0.	10.	10.	0.
0.91	15.	05.	20.	35.
2.91	5.	-25.	-5.	-5.
5.41	30.	25.	-10.	-10.
7.91	30.	25.	-10.	-5.
10.41	35.	35.	-10.	-10.

APPENDIX H

COMPUTER OUTPUT FOR MOHR RUPTURE ENVELOPE

BEST FIT PROGRAM.

MOHR DIAGRAM FOR MOHR CIRCLES.

COMPUTER OUTPUT FOR MOHR RUPTURE ENVELOPE

BEST FIT PROGRAM

The following pages show the program, test data and output for the Mohr rupture envelope program. After the word END, the data is printed line-by-line with the following format.

1.	Identification Number	Number of Tests
eg.	41	5
2.	Maximum Deviator Stress $\sigma_d(kg/cm^2)$	Cell pressure, $\sigma_3(kg/cm^2)$
eg.	2.17	0.00

The print-out of results on H-3 gives first the identification number, then the tangent of the angle the rupture envelope makes with the horizontal and the cohesion intercept. On succeeding lines are given the identification number, the shearing stress, normal stress and the 95% confidence limits at the average normal stress, one half the normal stress and twice the normal stress. All stress values are given in kgms/cm^2.

```
..I 920130 MOHR RUPTURE ENVELOPE
..LOAD FORGO CLOCK 600
      DIMENSION S1(10),S3(10),SS3(10),S1S3(10),SS1(10)
 5    R1=0.0
      R2=0.0
      R3=0.0
      R4=0.0
      R9=0.0
      READ,M,N
      DO 1 I=1,N
 1    READ,S1(I),S3(I)
      DO 2 I=1,N
      R1=R1+S3(I)
      R2=R2+S1(I)
      SS1(I)=S1(I)*S1(I)
      SS3(I)=S3(I)*S3(I)
      R9=R9+SS1(I)
      R3=R3+SS3(I)
      S1S3(I)=S1(I)*S3(I)
 2    R4=R4+S1S3(I)
      FN=N
      R5=R1*R1/FN
      R6=R1*R2/FN
      R7=R3-R5
      R8=R4-R6
      R10=R2*R2/FN
      R11-R9-R10
      R12=R11/R7
      R13=SQRT(R12)
      R14=(R13-1.0)/(2.0*SQRT(R13))
      R15=(R2-R13*R1)/(2.0*FN*SQRT(R13))
      PUNCH,M,R14,R15
      R16=R8/R7
      R17=SQRT(R16)
      R18=R7/FN
      R19A=(R13-R16)/(FN-2.)
      R19B=2.0*R18
      R19C=(R13+1.0)*(R13+1.0)/(R13*(R13+R16*R16))
      R20=(R2+R13*R1)/(FN*(R13+1.0))
      X1=R20
      Y1=R15+R14*x1
      R221=SQRT(R19A*R19B)
      PUNCH,M,Y1,X1,R221
      X2=0.0
      Y2=R15+R14*X2
      R222=SQRT(R19A*(R19B+R19C*(-R20)*(-R20)))
      PUNCH,M,Y2,X2,R222
      X3=2.0*R20
      Y3=R15+R14*X3
      R223=SQRT(R19A*(R19B+R19C*R20*R20))
      PUNCH,M,Y3,X3,R223
      GO TO 5
      END
```

DATA

```
 41      5
2.17    0.00
3.38    1.00
5.47    3.00
7.70    5.00
9.57    7.00
```

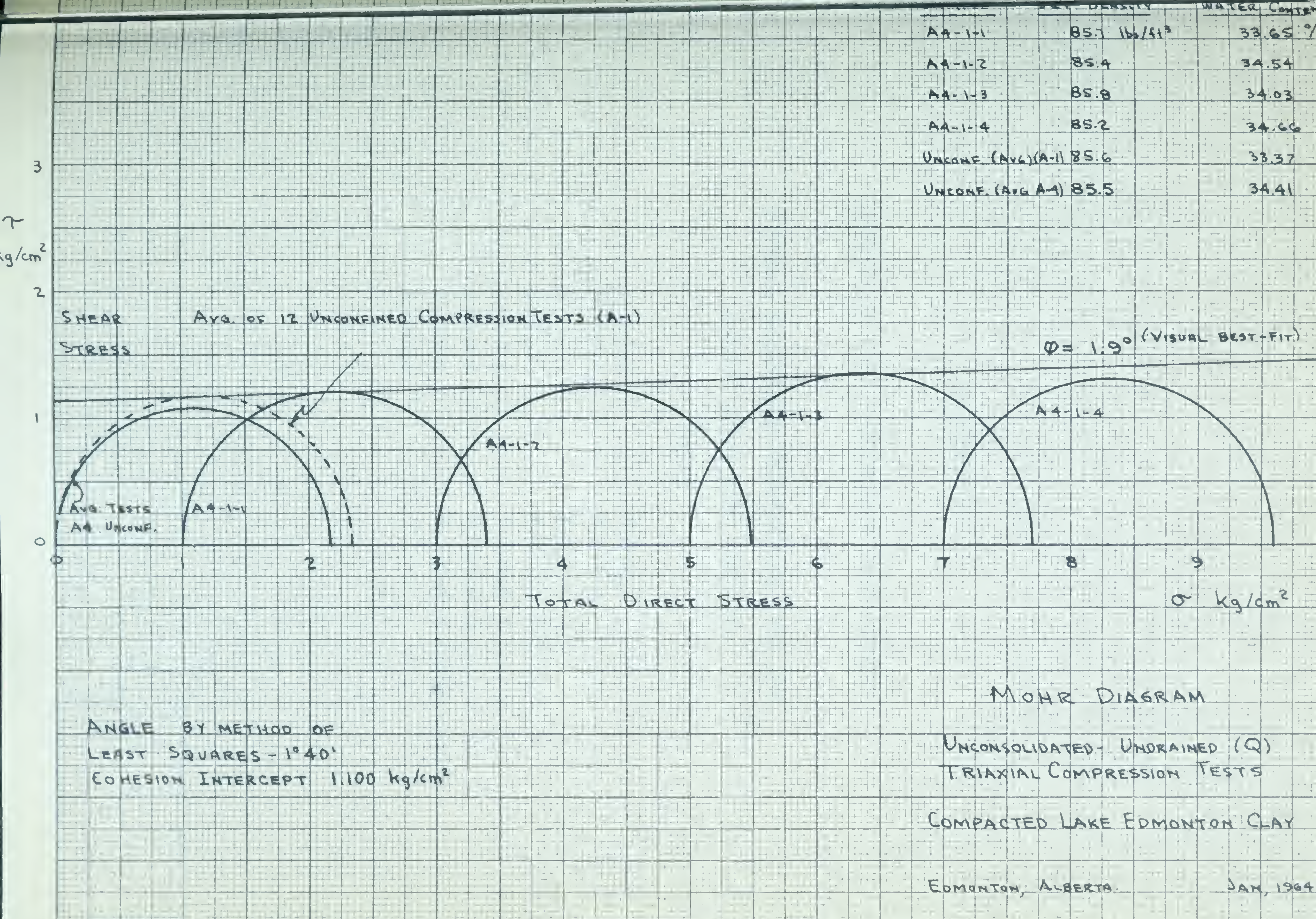
RESULTS

```
..LOAD FORGO CLOCK 600
          41  2.9208E-02  1.1002
          41  1.2285      4.3931     5.2673E-02
          41  1.1002      0.0000     1.0129E-01
          41  1.3568      8.7862     1.0129E-01

      ERROR LC-2 IN STATEMENT 0005 + 05 LINES
```

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 00 MINUTES,

14 SECONDS.


Water Content
A4-1-1 85.1 lbs/ft³ 33.65 %
A4-1-2 85.4 34.54
A4-1-3 85.8 34.03
A4-1-4 85.2 34.66
Unconf. (Avg)(A-1) 85.6 33.37
Unconf. (Avg A-4) 85.5 34.41
Shear Stress
τ kg/cm²
Avg. of 12 Unconfined Compression Tests (A-1)
Φ = 1.9° (Visual Best-Fit)
Avg. Tests A4 Unconf.
A4-1-1
A4-1-2
A4-1-3
A4-1-4
0
1
2
3
4
5
6
7
8
9
10
Total Direct Stress
σ kg/cm²
Angle by method of Least Squares - 1°40'
Cohesion Intercept 1.100 kg/cm²
Mohr Diagram
Unconsolidated - Undrained (Q) Triaxial Compression Tests
Compacted Lake Edmonton Clay
Edmonton, Alberta
Jan, 1964

APPENDIX I

SUMMARY OF SOIL COMPRESSION TEST RESULTS.

TYPICAL UNCONFINED COMPRESSION TEST AND DIRECT SHEAR TEST RESULTS.

<u>Tests</u>

A-1-3-1	F-2-Q_u
A-1-1-4	F-1-B-1
A-2-2-1	F-2-1-B 1/1
A-2-2-4	F-2-1-B 1/2
A-3-4-3	F-2-1-B 2
A-3-4-4	F-2-2-B 1
A-4-2-1	F-2-1-D S (soil only)
A-4-4-4	

Faculty of Graduate Studies

MODEL PILE TEST DATA

SOIL STRENGTH TEST RESULTS

Proving Ring No. 3-1584	Test Date SEPT. 22, 1963
Lateral Pressure σ_3 0 kg/cm^2	Base Diameter 2.0 In.
Avg. Specimen Size 3.65ø × 8.0 cm.	Net Shaft Length 9.85 In.

Sample Number	A1- 1-1	1-2	1-3	1-4	2-1	2-2	2-3	2-4	3-1	3-2	3-3	3-4	Avg	Shaft Avg	Soil Avg		Base Soil Avg.
Average Depth In.	1.75	5	8.75	13.5	1.75	5.25	9.	13.5	1.75	5.25	8.75	13.5	1.75	5.	9.	Over-all Avg.	13.5
σ_d kg/cm^2	2.28	2.39	2.45	2.66	2.27	2.26	2.34	2.31	2.15	-	2.28	2.30	2.23	2.33	2.36	2.30	2.42
$c = \tau_f$ kg/cm^2	1.14	1.19	1.12	1.33	1.13	1.13	1.17	1.15	1.07	-	1.14	1.15	1.12	1.16	1.18	1.15	1.21
w %	34.31	33.26	33.35	32.96	33.54	33.34	33.33	33.09	33.37	33.39	33.29	33.26	33.14	33.33	33.32	33.46	33.10
e	1.019	1.000	.9825	.9561	.998	.998	.964	.984	1.009	.993	1.003	1.008	1.009	.997	.983	.996	.983
γ_d lbs/ft^2	84.4	85.2	85.9	87.1	85.7	85.3	86.7	85.9	84.8	85.6	85.1	84.9	85.0	85.4	85.9	85.4	86.0
γ lbs/ft^2	113.3	113.5	114.6	115.8	114.4	113.7	115.6	114.3	113.1	114.2	113.4	113.1	113.6	113.8	114.5	114.0	114.0
S %	91.9	90.8	92.7	94.11	91.7	91.2	94.4	91.77	90.3	91.9	90.6	90.1	91.3	91.3	92.5	91.7	91.9
E kg/cm^2	156	126	108	86.1	95.7	53.0	44.0	30.1	87.3	-	37.0	27.6	113	†126	△108	115	*86.1
$\frac{E}{\tau_f} = \frac{E}{c}$	137	106	96.4	64.6	84.7	46.9	37.9	24.0	81.6	-	32.5	24.0	101	†106	△96.4	100	*64.7
ϵ_f %	4.5	5.0	4.5	6.5	7.0	8.5	9.0	11.0	6.0	-	9.0	10.5					
$\frac{\epsilon}{1-\epsilon}$ %	4.71	5.26	4.71	6.95	7.52	9.29	9.89	12.4	6.38	-	9.89	11.7					
$\dot{\epsilon}_f$ %/min	.446	.408	.400	.428	.422	.509	.429	.437	.42	-	.435	.434					

LEGEND:

$\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress

$\tau_f = \sigma_d/2 = c$ Shear Strength

w Moisture content

e Void ratio

γ_d Dry unit weight

γ Total unit weight

S Saturation

E Modulus of Elasticity

ϵ_f Unit Strain at failure

$\dot{\epsilon}$ Rate of strain at failure

Remarks: ESTIMATED STD. DEVIATION FOR MAXIMUM DEVIATOR STRESS IS ± .146 kg/cm^2; COEF. OF VARIATION ± 3.97 %, FOR SHAFT SOIL; FOR BASE SOIL STD. DEVIATION IS ± .208 kg/cm^2; COEF. OF VARIATION ± 8.6 %.

* TEST A-1-1-4 ONLY † TEST A-1-2-2 OMITTED

△ TEST A-1-1-3 ONLY.

THE UNIVERSITY OF ALBERTA
Faculty of Graduate Studies
MODEL PILE TEST DATA
SOIL STRENGTH TEST RESULTS

Proving Ring No. 3-1584	Test Number A2
Lateral Pressure σ_3 - 0 kg/cm^2	Test Date Oct. 11, 1963
Avg. Specimen Size 3.89 ϕ × 8.0 cm.	Base Diameter 1.0 In.
	Net Shaft Length 11.12 In.

Sample Number	A2- 1-1	1-2	1-3	1-4	2-1	2-2	2-3	2-4	3-1	3-2	3-3	3-4	AVG.	SHAFT AVG.	SOIL AVG.		BASE SOIL AVG.
Average Depth In.	2	6	10	14	2	6	10	14	2	6	10	14	2	6	10	OVER-ALL AVG.	14
σ_d kg/cm^2	3.84	3.51	3.72	5.12	3.74	3.83	3.82	4.66	3.52	3.72	3.85	4.68	3.58	3.69	3.80	3.69	4.82
$c = \tau_f$ kg/cm^2	1.74	1.75	1.86	2.56	1.87	1.91	1.91	2.33	1.76	1.86	1.92	2.34	1.79	1.84	1.90	1.84	2.41
w %	29.31	29.74	29.49	25.97	29.85	29.77	29.79	25.89	29.75	29.94	29.66	25.69	29.64	29.82	29.65	29.70	25.85
e	.909	.878	.868	.879	.884	.875	.870	.908	.895	.880	.874	.904	.896	.878	.871	.882	.897
γ_d lbs/ft^2	89.3	90.8	91.3	90.7	90.4	90.9	91.1	89.3	89.9	90.6	90.9	89.5	89.9	90.8	91.1	90.6	89.8
γ lbs/ft^2	115.5	117.8	118.2	114.2	117.4	117.9	118.3	112.4	116.6	117.7	117.8	112.4	116.5	117.8	118.1	117.5	113.1
S %	88.0	92.4	92.8	80.7	92.2	92.9	93.5	77.8	90.7	92.9	92.6	77.6	90.3	92.7	92.9	91.9	78.7
E kg/cm^2	189	116	149	235	153	138	152	235	176	286	236	235	173	180	179	177	235
$\frac{E}{\tau_f} = \frac{E}{c}$	109	66.3	80.2	91.7	81.9	72.3	79.5	101	100	154	123	101	96.6	97.8	94.2	96.1	97.5
ϵ_f %	7.0	8.5	5.0	4.0	5.5	6.5	5.5	3.5	5.0	6.0	5.5	3.5	5.8	7.0	5.3	6.1	3.3
$\frac{\epsilon}{1-\epsilon}$ %																	
$\dot{\epsilon}_f$ %/min	.394	.407	.370	.320	.373	.386	.373	.315	.373	.382	.377	.313					

LEGEND:
$\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress
$\tau_f = \sigma_d/2 = c$ Shear Strength
w Moisture content
e Void ratio
γ_d Dry unit weight
γ Total unit weight
S Saturation
E Modulus of Elasticity
ϵ_f Unit Strain at failure
$\dot{\epsilon}$ Rate of strain at failure

Remarks: Estimated Std. Deviation for Deviator Stress is: for shaft soil ± .146 kg/cm^2 Coef. of Variation is ± 3.97 % ; for base soil Std. Deviation is ± .26 kg/cm^2, Coef. of Variation ± 5.4 %.

UNIVERSITY OF ALBERTA

Faculty of Graduate Studies

MODEL PILE TEST DATA

SOIL STRENGTH TEST RESULTS

TEST NUMBER A-3

Test Date Oct 28, 1963

Base Diameter 2.0 In.

Net Shaft Length 10.75 In.

Proving Ring No. 3-1584

Lateral Pressure σ_3 0 ① kg/cm^2

Avg. Specimen Size 3.9⌀ × 8.0 cm.

Sample Number	A3-1-1	1-2	1-3	1-4	2-9	3-1	3-2 ①②	3-3 ③	3-4	4-1	4-2	4-3	4-4	AVG.	SHAFT SOIL AVG. ②	AVG.		BASE SOIL AVG.
Average Depth In.	2	6	10	14	14	2	6	10	14	2	6	10	14	2	6	10	OVER-ALL AVG.	14
σ_d kg/cm^2	3.47	3.97	4.04	5.16	5.41	3.88	4.89	4.25	5.13	3.96	4.11	4.30	5.50	3.76	4.04	4.20	4.00	5.30
$c = \tau_f$ kg/cm^2	1.73	1.98	2.02	2.58	2.70	1.94	2.44	2.12	2.66	1.98	2.05	2.15	2.75	1.88	2.02	2.10	2.00	2.65
w %	30.24	29.92	29.46	25.63	25.83	30.00	29.76	29.36	25.81	29.95	29.90	29.14	25.60	30.06	29.91	29.32	29.76	25.72
e	.905	.880	.863	.903	.878	.881	.871	.860	.899	.883	.874	.868	.881	.890	.877	.864	.877	.890
γ_d lbs/ft^2	89.4	90.6	91.4	89.5	90.7	90.6	91.1	91.6	89.7	90.5	90.9	91.2	90.6	90.2	90.7	91.4	90.8	90.1
γ lbs/ft^2	116.5	117.7	118.4	112.4	114.2	117.8	118.2	118.5	112.9	117.6	118.1	118.1	114.1	117.3	117.9	118.2	117.8	113.3
S %	91.2	92.8	93.2	77.5	80.3	93.0	93.3	93.2	78.4	92.6	93.3	92.9	80.5	92.2	93.1	92.6	92.6	78.9
E kg/cm^2	147	186	167	222	300	254	-	322	280	225	234	220	241	209	210	236	218	248
$\frac{E}{\tau_f} = \frac{E}{c}$	85.0	94.1	82.7	86.1	111	131	-	152	105	114	114	102	87.6	111	104	112	109	93.4
ϵ_f %	7.0	6.5	6.5	4.0	4.0	5.5	10.0	5.0	3.75	6.5	5.0	5.0	4.5	6.3	5.8	5.5	5.9	4.1
$\frac{\epsilon}{1-\epsilon}$ %																		
$\dot{\epsilon}_f$ %/min	.397	.392	.408	.325	.324	.382	.394	.357	.315	.382	.362	.357	.304	.387	.383	.383	.384	.317

LEGEND:

$\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress

$\tau_f = \sigma_d/2 = c$ Shear Strength

w Moisture content

e Void ratio

γ_d Dry unit weight

γ Total unit weight

S Saturation

E Modulus of Elasticity

ϵ_f Unit Strain at failure

$\dot{\epsilon}$ Rate of strain at failure

Remarks: ① For Test A 3-3-2 $\sigma_3 = 9\ kg/cm^2$ - Q Test.

② Test A3-3-2 not included in Avg. ③ Ends oiled.

For maximum deviator stress estimated standard deviation for shaft soil is ± .162 kg/cm^2, coef. of variation is ± 4.02 %; for base soil std. deviation is ± .183 kg/cm^2, coef of variation is ± 3.45 %

Faculty of Graduate Studies

MODEL PILE TEST DATA

SOIL STRENGTH TEST RESULTS

Proving Ring No. 3- 1584		Test Date	Dec 1963
Lateral Pressure σ_3	0 kg/cm^2	Base Diameter	1.0 In.
Avg. Specimen Size	3.9 × 8.0 cm.	Net Shaft Length	11.55 In.

Sample Number	A 4- 2-1	2-2	2-3	2-4	3-1	3-2	3-3	3-4	4-1	4-2	4-3	4-4	AVG.	SHAFT AVG.	SOIL AVG			BASE SOIL AVG.	
Average Depth In.	2	6	10	14	2	6	10	14	2	6	10	14	2	6	10	OVER-ALL AVG		14	
σ_d kg/cm^2	2.28	2.19	2.23	2.06	2.08	2.17	2.24	2.13	2.08	2.09	2.29	2.17	2.15	2.15	2.25	2.18		2.12	
c = τ_f kg/cm^2	1.14	1.10	1.12	1.03	1.04	1.09	112	1.07	1.04	1.05	1.15	1.09	1.08	1.08	1.13	1.09		1.06	
w %	34.33	34.63	34.05	34.52	34.30	34.71	34.30	34.64	34.20	34.36	33.56	34.40	34.28	34.56	34.30	34.38		34.52	
e	.978	1.00	.981	.989	1.00	1.01	.983	.982	1.03	1.01	.964	.980	1.00	1.01	.967	.997		.984	
γ_d lbs/ft^2	86.1	85.2	86.0	85.7	85.0	84.6	85.9	86.0	84.1	84.8	86.7	86.0	85.1	84.9	86.2	85.4		85.9	
γ lbs/ft^2	115.8	114.7	115.3	115.2	114.2	114.0	115.4	115.7	112.9	113.9	115.8	115.6	114.3	114.2	115.8	114.8		115.6	
S %	95.9	94.5	94.8	95.3	93.3	93.5	95.2	96.3	91.0	92.9	95.0	95.8	93.6	93.4	95.9	94.1		95.8	
E kg/cm^2	127	99.2	85.2	86.0	128	158	124	74.4	151	183	166	152	135	147	125	136		104	
$\frac{E}{\tau_f} = \frac{E}{c}$	111	90.0	76.0	83.3	123	145	111	69.5	145	174	144	139	125	136	111	125		98.1	
ϵ_f %	5.5	6.0	6.5	6.5	5.5	5.5	5.5	7.0	6.0	4.0	4.5	5.0	5.67	5.17	5.5	5.45		6.17	
$\frac{\epsilon}{1-\epsilon}$ %																			
$\dot{\epsilon}_f$ %/min	.410	.414	.417	.422	.415	.410	.407	.422	.417	.404	.398	.408	.414	.409	.407	.410		.417	

LEGEND:

$\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress

$\tau_f = \sigma_d/2 = c$ Shear Strength

w Moisture content

e Void ratio

γ_d Dry unit weight

γ Total unit weight

S Saturation

E Modulus of Elasticity

ϵ_f Unit Strain at failure

$\dot{\epsilon}$ Rate of strain at failure

Remarks: For Maximum Deviator Stress Estimated Standard Deviation for shaft soil is ± .081 kg/cm^2, Coef. of Variation ± 3.74 %; for base soil std. deviation is ± .056, Coef. of Variation is ± 2.63 %.

I-4

Proving Ring No. 783	Faculty of Graduate Studies	Test Date Dec 1963
Lateral Pressure σ_3 various kg/cm^2	MODEL PILE TEST DATA	Base Diameter 1.0 In.
Avg. Specimen Size 3.90 x 8.0 cm.	SOIL STRENGTH TEST RESULTS	Net Shaft Length 11.55 In.

Sample Number	A 4- 1-1	1-2	1-3	1-4			F-2	Direct Shear Test			Soil Based Mould	Properties on Dimensions			Total Aver.				
Average Depth In.	2	6	10	14								All Soils.			Shaft & Base				
σ_d kg/cm^2	2.38	2.47	2.70	2.57			2.17								2.17				
$c = \tau_f$ kg/cm^2	1.19	1.24	1.35	1.29			1.09	1.000							1.09				
w %	33.65	34.54	34.03	34.66			34.15	34.24				34.48			34.41				
e	.994	.996	.990	1.000			.997	1.057				1.000			.994				
γ_d lbs/ft^2	85.7	85.4	85.8	85.2			85.3	82.8				85.33			85.5				
γ lbs/ft^2	114.5	114.9	115.0	114.7			114.4	111.2				114.76			114.9				
S %	92.4	94.7	93.9	94.6			93.5	88.4				94.2			94.5				
E kg/cm^2	274	179	147	197			124								120				
$\frac{E}{\tau_f} = \frac{E}{c}$	230	144	109	165			114								111.6				
ϵ_f %	8.5	7.5	10.0	9.5			7.5								5.63				
$\frac{\epsilon}{1-\epsilon}$ %																			
$\dot{\epsilon}_f$ %/min	.200	.403	.417	-			.426								.412				

LEGEND: $\sigma_d = \sigma_1 - \sigma_3$ Max. Deviator Stress

$\tau_f = \sigma_d/2 = c$ Shear Strength

w Moisture content

e Void ratio

γ_d Dry unit weight

γ Total unit weight

S Saturation

E Modulus of Elasticity

ϵ_f Unit Strain at failure

$\dot{\epsilon}$ Rate of strain at failure

Remarks: Tests A4-1-1 to 1-4 are undrained-unconsolidated Q tests at following lateral pressures: 1-1 : 1 kg/cm^2; 1-2 : 3 kg/cm^2; 1-3 : 5 kg/cm^2; 1-4 : 7 kg/cm^2. By best-fit method $\tau_f = 1.100 + \sigma \tan 1°40'$.

UNIVERSITY OF ALBERTA DEPARTMENT OF CIVIL ENGINEERING OIL MECHANICS LABORATORY	PROJECT Thesis	
	SITE	
	SAMPLE A-1-1-4	
	LOCATION	
	HOLE	DEPTH 13.5"
	TECHNICIAN MCH	DATE Sept. 25/63

TRIAXIAL OR UNCONFINED COMPRESSION TEST

~~Pon~~	Strain Dial	Total Strain ~~(in)~~	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kg	Vertical Stress(PSI)	Time	Remarks	$\frac{\tau}{\tau_f}$
					10.55					
2	.11	5	.06		10.55	.22	.02			.008
8	.107	8	.1		10.56	.86	.08			.030
2	.105	16	.2		10.57	2.27	.21	0 35"		.079
5	0.101	23	.3		10.58	3.50	.33	50		.124
8	.0998	31	.4		10.59	4.78	.45	1' 05"		.17
60	996	38	.5		10.60	5.98	.56	1 20		.21
2	993	46	.6		10.61	7.15	.67	1 35		.25
02	986.	61	.8		10.63	9.06	.85	2 10		.32
9	.0978	77	1.0		10.66	10.66	1.00	2 40		.38
7	969	96	1.25		10.68	12.29	1.15	3 15		.433
6	955	115	1.5		10.70	13.93	1.30	3 50		.489
6	937	134	1.75		10.73	15.56	1.45	- -		.546
8	922	153	2.0		10.76	17.33	1.61	5-10		.606
8	911	172	2.25		10.79	18.96	1.76	5 40		.663
1	902	191	2.5		10.82	20.47	1.89	6-20		.711
6	894	210	2.75		10.85	22.00	2.03	6 55		.764
	889	229	3.0		10.88	23.18	2.13	7 30		.802
8	881	268	3.5		10.94	25.38	2.32	8-40		.873
	875	306	4.0		10.99	26.95	2.45	9-50		.922
	872	344	4.5		11.05	28.08	2.54	10-55		.956
	870	382	5.0		11.11	28.86	2.60	12-00		.978
	869	421	5.5		11.17	29.46	2.64	13 05		.993
	868	459	6.0		11.23	29.77	2.650	14 10		.997
9	867	497	6.5	$2\tau_f$	11.29	29.99	2.656	15 10	FAILURE	1.000
3	867	535	7.0		11.34	30.11	2.655	16 10		1.00
0	867	573	7.5		11.40	30.11	2.655	17 15		1.00
	869	612	8.0		11.47	29.46	2.57	18 05		.967

...e Geonor	Rate of Strain $\dot{\varepsilon}_f$ = .428 %/min = 25.7 %/hr
...ng Ring Code 3-1584	Lateral Pressure σ_3 0

SAMPLE DIMENSIONS	149.80	
Top In. (D1) 36.8	36.8	36.8
...e In. (D2) 36.6	36.6	36.6
...n In. (D3) 36.6	36.5	36.55
...ge In.		36.65
...ge Height 76.4	76.6	76.5
...ge Cross Section Area A = 10.55 In		

SAMPLE MOISTURE CONTENTS		START		END
Tare No.		A-19	A-20	
Wt. Tare & Moist Sample	G M.	103.99	105.92	209.91
Wt. Tare & Dry Sample	G M.	85.81	86.97	172.78
Wt. Tare	G M.	30.43	29.69	60.12
Wt. Moist Sample	G M.			149.79
Wt. Moisture	G M.	18.18	18.95	37.13
Wt. Dry Sample	G M.	55.38	57.28	112.66
Moisture Content	%	32.82	33.08	32.96

...ASUREMENTS AT END OF TEST

D1 :

D2 :

D3 :

Failure By:

60°

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test. 2.73 Assumed:		
Volume of Sample: C.C. 80.71 Cu. In.		
Volume of Soil Solids: C.C. 41.26	Void Ratio	.9561
Volume of Voids: C.C. 39.45		
Degree of Saturation: 94.11 %		
Density: Dry: 87.10 PCF. Moist: 115.8 PCF.		

SHEET 1/1

UNIVERSITY OF ALBERTA

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

PROJECT	THESIS		
SITE			
SAMPLE	A-1-3-1		
LOCATION			
HOLE		DEPTH	1.75"
TECHNICIAN	MCH.	DATE	SEPT 24/63

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d / ~~Bn~~	~~Strain Dial~~ k	Total Strain ~~(in)~~ cm	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kgs.	Vertical Stress (PSI)	TIME	Remarks	$\frac{\tau}{\tau_f}$
0	`	0			10.52	~~.55~~		0		
5	.109	6	.05		10.52	.55	.05			.02
3	.105	10 ~~8~~	.1		10.53	1.38	.13			.06
24	.103	16	.2		10.54	2.46	.23			.11
34	.101	24	.3		10.55	3.40	.32	55"		.15
42	.100	32	.4		10.56	4.20	.40	70"		.19
52	.0997	40	.5		10.57	5.18	.49	1-30		.23
62	.0995	48	.6		10.58	6.17	.58	-		.27
30	990	64	.8		10.60	7.92	.75	2-15		.35
98	983	80	1.0		10.63	9.63	.91	2-45		.42
0	.0973	100	1.25		10.65	11.67	1.10	3-35		.515
0	.0959	121	1.50		10.68	13.42	1.26	4 00		.583
5	942	141	1.75		10.70	14.99	1.40	4-40		.649
7	929	161	2.00		10.73	16.46	1.53	5-20		.710
1	921	181	2.25		10.75	17.57	1.63	5-55		.756
3	913	201	2.50		10.78	18.55	1.72	6-25		.797
5	908	221	2.75		10.81	19.51	1.80	7-05		.835
3	904	241	3.00		10.85	20.14	1.86	7-40		.863
8	897	282	3.50		10.91	21.35	1.96	8-50		.909
9	893	322	4.00		10.96	22.25	2.03	10-00		.942
1	891	362	4.5		11.02	22.88	2.08	11-10		.965
4	888	402	5.0		11.08	23.42	2.11	12-10		.979
9	886	442	5.5		11.14	23.83	2.139			.991
2	886	483	6.0	$2\tau_f$	11.19	24.10	2.154	14-25	FAILURE	1.000
5.2	886	523	6.5		11.25	24.21	2.152	15-30		.999
.5	886	563	7.0		11.31	24.06	2.13	16-30		.988
4	888	603	7.5		11.44	23.42	2.05	17-30		.951
		643	8.0							

ne	GEONOR	Rate of Strain	ε_f = 0.42%/min t_f = 14.4 min
ng Ring Code	3-1584	Lateral Pressure	σ_3 = 0

SAMPLE DIMENSIONS 153.42 153.38

Top In. (D_1)	36.60	36.68	36.64
e In. (D_2)	36.72	36.52	36.62
m In. (D_3)	36.60	36.48	36.54
age In.	80.40	80.36	36.60
age Height	80.38"		
age Cross Section Area A =	10.52 In.		

SAMPLE MOISTURE CONTENTS

		START (TOP)	START	END
Tare No.		A-11	A-12	
Wt. Tare & Moist Sample	G M.	101.08	112.67	213.75
Wt. Tare & Dry Sample	G M.	83.37	92.01	175.38
Wt. Tare	G M.	30.13	30.29	60.42
Wt. Moist Sample	G M.			153.33
Wt. Moisture	G M.	17.71	20.66	38.37
Wt. Dry Sample	G M.	53.24	61.72	114.96
Moisture Content	%	33.26	33.47	33.37

EASUREMENTS AT END OF TEST

D_1 :

D_2 :

D_3 :

75° 7/8" Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 84.56 Cu. In.

Volume of Soil Solids: C.C. 42.10 Void Ratio 1.009

Volume of Voids: C.C. 42.46

Degree of Saturation: 90.28 %

Density: Dry: 84.83 PCF. Moist: 113.1 PCF.

SHEET 1/1

UNIVERSITY OF ALBERTA

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A-2
SAMPLE	A-2-2-1
LOCATION	
HOLE	DEPTH 2"
TECHNICIAN MCH.	DATE OCT 17/63

TRIAXIAL OR UNCONFINED COMPRESSION TEST

~~/Rdg~~	~~Strain Dial~~ k	Total Strain ~~(in.)~~	Unit Strain	1 - Unit Strain	Corrected Area cm²	Load ~~(lbs.)~~ kg.	Vertical Stress(PSI)	TIME	Remarks	$\frac{\tau}{\tau_f}$
9	.107	4	.05		11.88	.96	.08			.02
7	.104	8	.1		11.89	1.78	.15	25"		.04
34	.1042	16	.2		11.90	3.40	.29	35		.08
52	.0997	24	.3		11.92	5.18	.43	0 55		.12
58	993	32	.4		11.93	6.75	.57	1' 15"		.15
33	989	40	.5		11.94	8.20	.69	1 35		.18
98	983	49	.6		11.95	9.63	.81	1 50		.22
38	961	65	.8		11.98	13.26	1.11	2 30		.30
00	915	81	1.0		12.00	18.30	1.53	3 15		.41
63	887	101	1.25		12.03	23.33	1.94	4 0		.52
10	875	121	1.5		12.06	27.13	2.25	5 35		.60
53	866	142	1.75		12.09	30.57	2.53	6 25		.68
34	862	162	2.0		12.12	33.10	2.73	7 0		.731
12	859	182	2.25		12.15	35.39	2.91			.779
		202	2.5		12.19	–		~~9 0~~		
30	8535	243	3.0		12.25	40.97	3.34	9 0		.893
06	852	283	3.5		12.31	43.11	3.50	10-10		.937
5	851	364	4.0		12.38	44.68	3.61	11 25		.966
8	850	364	4.5		12.44	45.73	3.676	12 30		.983
7	850	404	5.0		12.51	46.50	3.717	13 40		.995
30	849	444	5.5	$2\tau_f$	12.57	46.95	3.735	14 45	FAILURE	1.000
.8	849	485	6.0		12.64	47.10	3.726	15 50		.997
.2	849	525	6.5		12.71	47.14	3.71	16 55		.993
5	850	566	7.0		12.77	46.32	3.62	18 00		.968
9	850	606	7.5		12.84	46.64	3.63	19 00		.972
		647	8.0							
			8.5							
			9.0							

...ne GEONOR

...ng Ring Code 3-1584

Rate of Strain $\dot{e}_f$ = .373 %/min = 22.4%/hr

Lateral Pressure $\sigma_3 = 0$

SAMPLE DIMENSIONS

Top In. (D_1)	38.9	39.0	38.85
...e In. (D_2)	38.9	38.9	38.9
...n In. (D_3)	38.9	39.0	38.95
...ge In.			38.90
...ge Height	80.9	80.9	80.9
...ge Cross Section Area A =			11.88 In.

...ASUREMENTS AT END OF TEST

D_1 :

D_2 :

D_3 :

Failure By:

45°

Corrected Area = $\frac{A}{1 - \text{Strain}}$

SAMPLE MOISTURE CONTENTS

			START		END
Tare No.		A 15	A 16		
Wt. Tare & Moist Sample	G M.	117.23	124.35		241.58
Wt. Tare & Dry Sample	G M.	97.11	102.90		200.01
Wt. Tare	G M.	29.98	30.78		60.76
Wt. Moist Sample	G M.				180.82
Wt. Moisture	G M.	20.12	21.45		41.57
Wt. Dry Sample	G M.	67.13	72.12		139.25
Moisture Content	%	29.97	29.74		29.85

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 96.11 Cu. In.

Volume of Soil Solids: C.C. 51.01

Volume of Voids: C.C. 45.10

Void Ratio .8841

Degree of Saturation: 92.17 %

Density: Dry: 90.41 PCF. Moist: 117.4 PCF.

SHEET 1 OF 1.

PROJECT	THESIS
SITE	TEST A2
SAMPLE	A2-2-4
LOCATION	
HOLE	DEPTH 14"
TECHNICIAN MCH.	DATE OCT. 17/63

UNIVERSITY OF ALBERTA

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d / ~~Pon~~	~~Strain Dial~~ k	Total Strain ~~(in.)~~	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kg	Vertical Stress(PSI)	TIME	Remarks	$\frac{\tau}{\tau_f}$
3	.11	4	.05		11.88	.33	.03			.01
1	.107	8	.1		11.89	1.17	.10			.02
30	.102	16	.2		11.90	3.06	.26			.06
54	.0997	24	.3		11.92	5.38	.45	55		.10
82	989	32	.4		11.93	8.11	.68	1' 15"		.15
13	976	40	.5		11.94	11.03	.92	1 40		.20
54	947	48	.6		11.95	14.58	1.22	2- 0		.26
44	895	64	.8		11.98	21.84	1.82			.39
26	871	80	1.0		12.00	28.40	2.36	3 50		.51
09	859	100	1.25		12.03	35.13	2.92	4 50		.627
18	853	120	1.5		12.06	40.77	3.38	5 40		.726
30	8505	140	1.75		12.09	45.08	3.73	6 30		.800
73	848	161	2.0		12.12	48.60	4.01	7- 20		.860
04	847	181	2.25		12.15	51.16	4.21	8- 00		.903
30	846	201	2.5		12.19	53.23	4.37	8 40		.937
50	845	221	2.75		12.21	54.93	4.49	9 20		.964
64	844	241	3.0		12.25	56.04	4.575	9 55		.982
81	843	281	3.5	$2\tau_f$	12.31	57.40	4.664	11 05	FAILURE	1.000
84.3	843	321	4.0		12.38	57.69	4.659	12-15	Max 684.6 @ 310	.999
53	845	361	4.5		12.44	55.13	4.43	13 00		.952
		401	5.0							
		441	5.5							
		482	6.0							
			6.5							
			7.0							
			7.5							
			8.0							
			8.5							

...ine	GEONOR	Rate of Strain	$\dot{\varepsilon}_f$ = .315 %/min = 18.9%/hr
...ing Ring Code	3-1584	Lateral Pressure	$\sigma_3 = 0$

SAMPLE DIMENSIONS			
Top In. (D_1)	38.8	39.0	38.9
...le In. (D_2)	38.9	39.1	39.0
...om In. (D_3)	38.8	38.9	38.85
...age In.			38.9
...age Height	80.2	80.3	80.25
...age Cross Section Area A = 11.88 In.			

SAMPLE MOISTURE CONTENTS		START	END
Tare No.		—	
Wt. Tare & Moist Sample	G M.	175.50	
Wt. ~~Tare &~~ Dry Sample	G M.		
Wt. Tare	G M.	3.80	
Wt. Moist Sample	G M.	171.70	
Wt. Moisture	G M.	35.31	
Wt. Dry Sample	G M.	136.39	
Moisture Content	%	25.89	

...EASUREMENTS AT END OF TEST

D_1 :

D_2 :

D_3 :

Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 95.34 Cu. In.

Volume of Soil Solids: C.C. 49.96

Volume of Voids: C.C. 45.38

Void Ratio .9083

Degree of Saturation: 77.82%

Density: Dry: 89.26 PCF. Moist: 112.4 PCF.

UNIVERSITY OF ALBERTA DEPARTMENT OF CIVIL ENGINEERING SOIL MECHANICS LABORATORY	PROJECT THESIS ~~SITE~~ TEST A3 SAMPLE A3-3-4 LOCATION HOLE DEPTH 14" TECHNICIAN MCH DATE Nov. 3, 1963

TRIAXIAL OR UNCONFINED COMPRESSION TEST

Load Dial ~~Rdg~~	~~Strain Dial~~ k	Total Strain ~~(in)~~	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kg	σ_d Vertical Stress ~~(PSI)~~	TIME	Remarks	$\frac{\tau}{\tau_f}$
6	.108	4	.05		11.88	0.64	.05			.01
13	.106	8	.1		11.89	1.38	.12			.02
34	.101	16	.2		11.90	3.40	.29	0'-35"		.06
64	.0995	24	.3		11.92	6.39	.53			.10
93	985	32	.4		11.93	9.16	.77			.15
122	972	40	.5		11.94	11.8.58	.99	1 40		.19
155	946	48	.6		11.95	14.66	1.23			.24
244	895	64	.8		11.98	21.84	1.82			.354
329	871	80	1.0		12.00	28.66	2.39	3 50		.466
419	858	100	1.25		12.03	35.95	2.99			.582
493	853	121	1.5		12.06	42.05	3.49	5-40		.680
549	8495	141	1.75		12.09	46.64	3.86			.752
595	847	161	2.0		12.12	50.40	4.16	7-20		.810
637	845	181	2.25		12.15	53.83	4.43			.864
670	844	201	2.5		12.19	56.55	4.64	8 45		.903
698	843	221	2.75		12.21	58.84	4.82			.938
-		241	3.0		12.25	-	-			
735	841	261	3.25		12.28	61.81	5.03	10 40		.981
748	841	281	3.5		12.31	62.90	5.11			.995
754	840.5	301	3.75		12.34	63.34	5.132	11 55	FAILURE	1.000
756	840	321	4.0		12.38	63.50	5.129	12 05		.998
705	842	362	4.5		12.44	59.36	4.77	13 15		.929
		402	5.0							
		442	5.5							
		482	6.0							
		522	6.5							
		562	7.0							
		602	7.5							

Machine GEONOR

Proving Ring Code 3-1584

Rate of Strain $\dot{\varepsilon}_t$ = .315 %/min = 18.9 %/hr

Lateral Pressure $\sigma_3 = 0$

SAMPLE DIMENSIONS		wt 172.99 172.90	
Dia. Top In. (D1)	39.1	38.8	38.95
Middle In. (D2)	38.9	38.9	38.9
Bottom In. (D3)	38.9	38.9	38.9
Average In.			38.9
Average Height	80.3	80.4	80.35
Average Cross Section Area A = 11.88 In.			

SAMPLE MOISTURE CONTENTS		START		END
Tare No.		A56 B	A57 T	TOTAL
Wt. Tare & Moist Sample	G M.	133.94	99.94	233.88
Wt. Tare & Dry Sample	G M.	112.82	85.64	198.46
Wt. Tare	G M.	31.02	30.19	61.21
Wt. Moist Sample	G M.			172.67
Wt. Moisture	G M.	21.12	14.30	35.42
Wt. Dry Sample	G M.	81.80	55.45	137.25
Moisture Content	%	25.82	25.79	25.81

MEASUREMENTS AT END OF TEST

D_1 :

D_2 :

D_3 :

Failure By:

Corrected Area $= \frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 95.46 Cu. In.

Volume of Soil Solids: C.C. 50.27

Volume of Voids: C.C. 45.19

Void Ratio .8989

Degree of Saturation: 78.38 %

Density: Dry: 89.72 PCF. Moist: 112.9 PCF.

SHEET 1 OF 1

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A3
SAMPLE	A3-4-3
LOCATION	
HOLE	DEPTH 10"
TECHNICIAN MCH	DATE Nov 2/63

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d	~~Strain Dial~~ k	Total Strain ~~(in)~~	Unit Strain	1 - Unit Strain	Corrected Area	Load ~~(lbs.)~~ kgs.	Vertical Stress(PSI)	Time	Remarks	$\frac{\tau}{\tau_f}$
10	.107	4	.05		11.82	1.07	.09			.02
20	104	8	.1		11.83	2.06	.17			.04
40	.100	16	.2		11.84	4.00	.34	0-40		.08
63	.0995	24	.3		11.85	6.27	.53			.12
85	988	32	.4		11.86	8.40	.71			.17
13	976	40	.5		11.87	11.03	.93	1'-40"		.22
53	948	48	.6		11.88	14.50	1.22			.284
24	903	64	.8		11.91	20.23	1.70			.396
03	877	80	1.0		11.94	26.57	2.22	3-40		.493
86	862	100	1.25		11.97	33.27	2.78			.647
47	856	120	1.5		12.00	38.26	3.19	5 25		.743
91	853	140	1.75		12.03	41.88	3.48			.810
22	851	160	2.0		12.06	44.42	3.68	6 55		.856
16	850	180	2.25		12.09	46.41	3.84			.916
64	849	200	2.5		12.12	47.97	3.96	8 10		.921
79	848	220	2.75		12.14	49.09	4.04			.940
99	847.5	240	3.0		12.19	49.92	4.10	9 25		.953
08	847	280	3.5		12.25	51.50	4.20	10 35		.977
9	846	320	4.0		12.32	52.37	4.25	11 45		.989
27	846	360	4.5		12.38	53.04	4.28	12 50		.996
3.0	845	401	5.0		12.45	53.52	4.298	14 00	FAILURE	1.000
6.5	845	441	5.5		12.51	53.74	4.295	15 00		.999
7.0	845	481	6.0		12.58	53.83	4.28	16 05		.996
1.0	8455	521	6.5		12.64	53.35	4.22	17 05		.981
0	850	561	7.0		12.71	45.90	3.61	17 40		.841
		601	7.5		12.78					
		641	8.0		12.85					
		681	8.5		12.92					

ine	GEONOR	Rate of Strain	$\dot{\varepsilon}_f$ = .357 %/min = 21.4 %/hr
ing Ring Code	3-1584	Lateral Pressure	$\sigma_3 = 0$

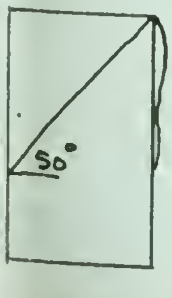
SAMPLE DIMENSIONS .wt. 179.29 / 179.18

Top In. (D1)	38.6	38.9	38.75
le In. (D2)	38.7	38.9	38.8
om In. (D3)	38.7	38.9	38.8
age In.			38.8
age Height	80.1	80.1	80.1

age Cross Section Area A = 11.82 In.

SAMPLE MOISTURE CONTENTS		START		END	
Tare No.		A50	A51	TOTAL	
Wt. Tare & Moist Sample	G M.	106.96	131.63	238.59	
Wt. Tare & Dry Sample	G M.	89.62	108.66	198.28	
Wt. Tare	G M.	30.02	29.91	59.93	
Wt. Moist Sample	G M.			178.66	179.2
Wt. Moisture	G M.	17.34	22.97	40.31	40.8
Wt. Dry Sample	G M.	59.60	78.65	138.35	138.3
Moisture Content	%	29.09	29.21	29.14	⊕ 29.5

EASUREMENTS AT END OF TEST



D1 :
D2 :
D3 :
Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity:	From Test: 2.73	Assumed:	
Volume of Sample:	C.C.	94.68 Cu. In.	
Volume of Soil Solids:	C.C. 50.68	Void Ratio	.8682
Volume of Voids:	C.C. 44.00		
Degree of Saturation:	92.92 %		
Density:	Dry: 91.18 PCF.	Moist: 118.1 PCF.	

⊕ Value used.

SHEET 1 OF 1

UNIVERSITY OF ALBERTA

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A3
SAMPLE	A3-4-4
LOCATION	
HOLE	DEPTH 14"
TECHNICIAN MCH	DATE Nov 2, 1963

TRIAXIAL OR UNCONFINED COMPRESSION TEST

~~/Pon~~	~~Strain Dial~~ k	Total Strain ~~(in)~~	Unit Strain	1 - Unit Strain	Corrected Area cm²	Load ~~(lbs.)~~ kg	Vertical Stress ~~(PSI)~~	TIME	Remarks	$\frac{\tau}{\tau_f}$
6	.108	4	.05		11.82	.64	.05			.01
4	.105	8	.1		11.83	1.48	.12			.02
3	.101	16	.2		11.84	3.33	.28	35"		.05
7	.0996	24	.3		11.85	5.68	.48			.09
3	988	32	.4		11.86	8.20	.69			.13
4	976	40	.5		11.87	11.13	.94	1'-40"		.17
	958	48	.6		11.88	13.51	1.14			.208
7	907	64	.8		11.91	19.68	1.65			.301
5	879	80	1.0		11.94	25.93	2.17	3-35		.395
3	861	100	1.25		11.97	33.84	2.83			.515
4	854	120	1.5		12.00	40.48	3.37	5 35		.613
	850	141	1.75		12.03	45.65	3.79			.690
0	8475	161	2.0		12.06	50.00	4.15	7 15		.755
6	845	181	2.25		12.09	53.74	4.49			.817
	844	201	2.5		12.12	56.80	4.69	8 45		.853
	842	221	2.75		12.14	59.61	4.91			.893
	841	241	3.0		12.19	61.73	5.064	10 10		.918
	840	261	3.25		12.22	63.59	5.20			.946
	839.5	281	3.5		12.25	65.15	5.318	11 25		.968
	839	301	3.75		12.28	66.20	5.390			.982
		321	4.0		12.32	—	—	—		
	838	361	4.5		12.38	68.05	5.496	13 50	FAILURE	1.000
5	838	402	5.0		12.45	68.09	5.469	14 50	May 815	.994
7	841	442	5.5		12.51	61.39	4.91	15 30		.894
		482	6.0							
		522	6.5							
		562	7.0							
		602	7.5							

...ne GEONOR — Rate of Strain $\dot{\varepsilon}_f$ = .304 %/min = 18.2%/hr.

...ng Ring Code 3-1584 — Lateral Pressure $\sigma_3 = 0$

SAMPLE DIMENSIONS (wt 173.67, 173.61)

Top In. (D_1)	38.3	39.3	39.8
...e In. (D_2)	38.3	39.3	39.8
...m In. (D_3)	38.3	39.3	39.8
...ge In.			
...ge Height	80.4	80.3	80.35

...ge Cross Section Area A = 11.82 In.

SAMPLE MOISTURE CONTENTS

			START	END	
Tare No.		AS2 B	AS3 T		
Wt. Tare & Moist Sample	G M.	109.87	123.97	233.84	
Wt. Tare & Dry Sample	G M.	93.68	104.88	198.56	
Wt. Tare	G M.	30.39	30.34	60.73	
Wt. Moist Sample	G M.			173.11	173.64
Wt. Moisture	G M.	16.19	19.09	35.28	35.81
Wt. Dry Sample	G M.	63.29	74.54	137.83	137.83
Moisture Content	%	25.58	25.61	25.60	25.98*

...ASUREMENTS AT END OF TEST

65°

65°

D_1 :

D_2 :

D_3 :

Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

Specific Gravity: From Test: 2.73 Assumed:

Volume of Sample: C.C. 94.97 Cu. In.

Volume of Soil Solids: C.C. 50.49 — Void Ratio .8810

Volume of Voids: C.C. 44.48

Degree of Saturation: 80.51 %

Density: Dry: 90.56 PCF. Moist: 114.1 PCF.

* Value used.

SHEET 1 OF 1

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A 4
SAMPLE	A4-2-1
LOCATION	
HOLE	DEPTH 2"
TECHNICIAN MCH	DATE DEC 11/63

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d / Pon	Strain Dial	Total Strain ~~(in.)~~	Unit Strain	1 - Unit Strain	Corrected Area cm^2	Load ~~(lbs.)~~	Vertical Stress(PSI)	Time (Min)	Remarks	$\frac{\tau}{\tau_f}$
					11.88					
6		4	.05		11.88	.64	.05			.02
11		8	.1		11.89	1.17	.0.10			.04
24		16	.2		11.90	2.46	.20	0 30		.09
10		24	.3		11.92	4.00	.33			.15
8		40	.5		11.94	7.72	.65	1 30		.29
9		60	.75		11.97	11.58	.97			.43
52		80	1.0		12.00	14.42	1.20	3 0		.527
31		100	1.25		12.03	16.78	1.39			.610
05		120	1.5		1206	18.72	1.55	4 - 15		.681
25		140	1.75		12.09	20.32	1.68			.738
13		161	2.0		12.12	21.75	1.79	5 30		.786
55		181	2.25		12.15	2272	1.87			.821
39		201	2.5		12.19	23.83	1.95	6 35		.856
30		221	2.75		12.21	24.72	2.02			.886
9		241	3.0		12.25	25.44	2.08	7 50		.913
4		281	3.5		12.31	26.64	2.16	9 0		.949
4		321	4.0		12.38	27.44	2.22	10 5		.975
0		361	4.5		12.44	27.92	2.24$_5$	11 10		.986
50		402	5.0		12.51	28.33	2.26$_4$	12 25		.995
5		442	5.5		12.57	28.61	2.27$_6$	13 25	FAILURE	1.000
.0		482	6.0		12.64	28.73	2.27$_2$	14 30		.999
3		522	6.5		12.71	28.75	2.26$_1$	15 35		.995
		562	7.0		12.77	28.40	2.22	16 40		.975
)		602	7.5		12.84	27.04	2.11	17 35		.926
		642	8.0		12.91					
		683	8.5		12.98					
		723	9.0		13.07					

ne GEONOR — Rate of Strain $\dot{\varepsilon}_f$ = .410 % / min = 24.6 % / hr

ng Ring Code 3-1584 — Lateral Pressure $\sigma_3 = 0$

SAMPLE DIMENSIONS — wt 177.16, 177.08

Top In. (D_1)	39.0	39.1	39.05
e In. (D_2)	38.9	38.9	38.9
m In. (D_3)	38.8	38.9	38.85
ge In.			38.9
ge Height	80.3	80.3	80.3

ge Cross Section Area A = 11.88 In.

A-24 w_f = 34.97%

EASUREMENTS AT END OF TEST

40° PENETRATION

D_1 : .

D_2 : 48, 51, 51

D_3 :

Failure By:

24.0°C

Corrected Area = $\frac{A}{1 - \text{Strain}}$

SAMPLE MOISTURE CONTENTS

		START		END
Tare No.		A49	A50	TOTAL
Wt. Tare & Moist Sample	G M.	124.13	113.23	237.36
Wt. Tare & Dry Sample	G M.	100.27	91.88	
Wt. Tare	G M.	30.41	30.03	60.44
Wt. Moist Sample	G M.			176.92
Wt. Moisture	G M.	23.86	21.35	45.21
Wt. Dry Sample	G M.	69.86	61.85	131.71
Moisture Content	%	34.15	34.51	34.33

Specific Gravity: From Test: 2.73 Assumed: γ_w = 62.40

Volume of Sample: C.C. 95.40 Cu. In.

Volume of Soil Solids: C.C. 48.24 — Void Ratio 0.9776

Volume of Voids: C.C. 47.16

Degree of Saturation: 95.87 %

Density: Dry: 86.09 PCF. Moist: 115.8 PCF.

SHEET 1 OF 1

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST A 4
SAMPLE	A 4 - 4 - 4
LOCATION	
HOLE	DEPTH 14"
TECHNICIAN MCH	DATE Dec 10, 1963

TRIAXIAL OR UNCONFINED COMPRESSION TEST

ad al ~~(Lbs)~~	Strain Dial	Total Strain (in)	Unit Strain %	1 - Unit Strain	Corrected Area cm^2	Load ~~(lbs)~~ kg	Vertical Stress ~~(PSI)~~	TIME (MIN	Remarks	$\frac{\tau}{\tau_f}$
					11.79					
9	4		.05		11.79	.96	.08			.04
5	8		.1		11.80	1.58	.13			.06
1	16		.2		11.81	3.16	.27	0 - 35"		.12
1	24		.3		11.83	5.08	.43			.20
7	40		.5		11.85	8.60	.73			.34
2	59		.75		11.87	12.74	1.07	2 - 20		49
6	79		1.0		11.91	15.56	1.31	3 00		.603
5	99		1.25		11.94	17.90	1.50	3 40		.691
8	119		1.5		11.97	19.76	1.65			.761
6	139		1.75		12.00	21.18	1.77	5 - 00		.816
9	159		2.0		12.03	22.25	1.84	5 - 35		.848
0	179		2.25		12.06	23.11	1.91			.880
0	199		2.5		12.10	23.92	1.97	6 - 45		.908
7	219		2.75		12.12	24.49	2.02			.931
73	238		3.0		12.16	24.97	2.05			.946
02	278		3.5		12.21	25.70	2.10	9 00		.969
9	318		4.0		12.29	26.23	2.13	10 05		.981
4.2	358		4.5		12.34	26.65	2.15_9	11 10		.994
1.5	397		5.0		12.41	26.92	2.16_9	12 15	FAILURE	1.000
9.0	437		5.5		12.47	27.04	2.16_8	13 15		.999
0.3	477		6.0		12.54	27.15	2.16_5	14 20		.998
0.0	517		6.5		12.62	27.13	2.14_9	15 25		.991
3.	556		7.0		12.67	26.57	2.10	16 25		.968

hine GEONOR — Rate of Strain $\dot{\varepsilon}_1$ = .408 % / min = 24.5 % / hr.

ving Ring Code 3 - 1584 — Lateral Pressure σ_3 = 0

SAMPLE DIMENSIONS — Wt 173.46, 173.37

Top In. (D_1)	38.6	38.8	38.7
lle In. (D_2)	38.7	38.8	38.75
om In. (D_3)	38.6	38.9	38.75
rage In.			38.75
rage Height	79.4	79.4	79.4

rage Cross Section Area A = 11.79 ~~In.~~

w = A-8 34.76 %

EASUREMENTS AT END OF TEST

Penetration

D_1 :
D_2 : 51, 52, 50
D_3 :
Failure By:

Corrected Area = $\frac{A}{1 - \text{Strain}}$

SAMPLE MOISTURE CONTENTS

		START		END
Tare No.		A9	A10	
Wt. Tare & Moist Sample	G M.	110.40	123.07	233.47
Wt. Tare & Dry Sample	G M.	89.92	99.32	
Wt. Tare	G M.	30.36	29.82	60.18
Wt. Moist Sample	G M.			173.29
Wt. Moisture	G M.	20.48	23.75	44.29
Wt. Dry Sample	G M.	59.56	69.50	129.06
Moisture Content	%	34.38	34.17	34.27

Specific Gravity: From Test: 2.73 Assumed: ω_o % = 34.4

Volume of Sample: C.C. 93.61 Cu. In.

Volume of Soil Solids: C.C. 47.27 — Void Ratio 0.9803

Volume of Voids: C.C. 46.34

Degree of Saturation: 95.79 %

Density: Dry: 86.03 PCF. Moist: 115.6 PCF.

UNIVERSITY OF ALBERTA

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

PROJECT	THESIS
~~SITE~~	TEST F-2-Qu
SAMPLE	
LOCATION	
HOLE	DEPTH
TECHNICIAN MCH	DATE JAN 18, 1964

TRIAXIAL OR UNCONFINED COMPRESSION TEST

d ~~/Pan~~	Strain Dial	Total Strain ~~(in)~~	Unit Strain	I - Unit Strain	Corrected Area	Load (lbs.)	Vertical Stress(PSI)	TIME (MIN)	Remarks
					11.82				
4	4		.05		11.82	.44	.04		
8	8		.1		11.83	.88	.08		
7	16		.2		11.84	1.78	.15	0 30	
26	24		.3		11.85	2.65	.22		
37	40		.5		11.87	3.70	.31	1 15	
69	60		.75		11.90	6.85	.58		
09	80		1.0		11.94	10.66	.89	2 40	
18	100		1.25		11.97	14.09	1.18		
33	120		1.5		12.00	16.93	1.41	4 - 10	
08	140		1.75		12.03	18.96	1.58		
26	160		2.0		12.06	20.40	1.69	5 25	
11	179		2.25		12.09	21.60	1.79		
3	199		2.5		12.12	22.56	1.86		
1	219		2.75		12.14	23.18	1.91	~~7 40~~	
68	239		3.0		12.19	23.76	1.95	7 40	
30	279		3.5		12.25	24.72	2.02	-	
8	319		4.0		12.32	25.38	2.06	10 00	
05	359		4.5		12.38	25.92	2.09	11 05	
01	399		5.0		12.45	26.40	2.12	12 10	
6_0	438		5.5		12.51	26.81	2.14_3	13 15	
0_0	478		6.0		12.58	27.13	2.15_6	14 20	
2_5	518		6.5		12.64	27.33	2.16_2	15 25	
4_3	558		7.0		12.71	27.46	2.16_0	16 30	
6_9	598		7.5		12.78	27.67	2.165	17 35	FAILURE
8_2	638		8.0		12.85	27.78	2.16_1	18 40	
9_0	677		8.5		12.92	27.84	2.15_4	19 45	
9_1	717		9.0		13.00	27.85	2.14_2	~~1~~20 40	

ne	GEONOR	Rate of Strain	$\dot{\epsilon}_t = 0.426\%/min = 25.6\%/hr.$
ng Ring Code	3-1584	Lateral Pressure	$\sigma_3 = 0$

SAMPLE DIMENSIONS

Top In. (D1)	38.1	38.8	38.15
e In. (D2)	38.9	38.9	38.85
m In. (D3)	38.8	38.9	38.85
ge In.			38.8
ge Height	79.7	79.7	79.7

ge Cross Section Area A = 11.82 In.

ALSO A 10 W BEFORE TRIMMING = 34.69%

ASUREMENTS AT END OF TEST

INITIAL TOTAL WT 172.83

D_1 : 172.73

D_2 :

D_3 :

Failure By:

Corrected Area = $\frac{A}{1 - Strain}$

SAMPLE MOISTURE CONTENTS		START		END
Tare No.		A 11 B	A 12 T	TOTAL
Wt. Tare & Moist Sample	G M.	114.25	118.97	233.22
Wt. Tare & Dry Sample	G M.	92.87	96.37	
Wt. Tare	G M.	30.15	30.30	60.45
Wt. Moist Sample	G M.			172.77
Wt. Moisture	G M.	21.38	22.60	43.98
Wt. Dry Sample	G M.	62.72	66.07	128.79
Moisture Content	%	34.08	34.20	34.15

Specific Gravity: From Test: 2.73 Assumed: $\gamma_w = 62.40 \#/ft^3$

Volume of Sample: C.C. 94.21 Cu. In.

Volume of Soil Solids:	C.C. 47.17	Void Ratio	.9972
Volume of Voids:	C.C. 47.04		

Degree of Saturation: 93.49 %

Density: Dry: 85.30 PCF. Moist: 114.4 PCF.

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE B1	TRIAL NO. 1
LOCATION	
HOLE	DEPTH
TECHNICIAN MCH	DATE DEC 9, 1964

SHEAR BOX:	PLATE	INITIAL GROSS WEIGHT	NOTE: SOIL DATA TAKEN FROM AVG. FOR BASE SOIL IN TEST A4	SAMPLE DESCRIPTION SMOOTH BRASS PLATE PRESSED ONTO SURFACE OF LAKE EDMONTON CLAY AS COMPACTED FOR TEST A4 IN PILE TEST MOULD. FAILURE OCCURRED AT SOIL - PLATE INTERFACE.
WIDTH	6.002 cm	FINAL GROSS WEIGHT		
LENGTH	6.000	WEIGHT OF SPECIMEN		
INITIAL AREA	36.01 cm^2	VOID RATIO, e = 0.984 ; w = 34.52%		
HEIGHT OF SPECIMEN	SOIL 1.50 cm	TOTAL NORMAL LOAD	37.03 kg	Gs 2.73 ~~ttt~~ S = 95.8% ~~ttt~~ f_{max} = .257 kg/cm^2
VOLUME OF SPECIMEN	54.0 cm^3	UNIT NORMAL LOAD σ	1.03 kg/cm^2	~~Cu~~ γ = 115.6 #/ft^3 γ_d = 85.9 #/ft^3 τ_f = 1.06 kg/cm^2

Horizontal Base Movement: Turns	Horizontal Base Movement: Inches	Proving Ring Dial Reading .0001 in.	Vertical Dial Reading In.	Vertical Strain	Total Horizontal Shear Strain In.	Sample Length cm	Cross-Section Area cm^2	Total Shear Force kg	Shear Stress f kg/cm^2	$\frac{f}{f_{max}}$
	.0000	0	.2482		0	6.000	36.01			
	.0100	25			.0075	5.99	35.95	9.12	0.254	.958
	.0200	26			.0174	5.98	35.88	9.50	.265	1.000
	.0300	24			.0276	5.97	35.82	8.75	.244	.921
	.1000	22	.2482		.9978	5.90	35.40	8.02	.227	.857

SAMPLE F2-1-B1 TRIAL 1 SHEET 1 OF 2

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 1-B1 TRIAL NO. 1
LOCATION
HOLE DEPTH
TECHNICIAN M.C.H DATE JAN 18, 1964

SHEAR BOX:	PLATE	SOIL
WIDTH	6.002 cm	5.93 cm
LENGTH	6.000	6.02
INITIAL AREA	36.01 cm²	35.70 cm²
HEIGHT OF SPECIMEN		.0700 in = 1.78 cm
VOLUME OF SPECIMEN		63.55 cm³

INITIAL GROSS WEIGHT	145.00
FINAL GROSS WEIGHT	30.34
WEIGHT OF SPECIMEN	114.66
VOID RATIO, e	= 1.032, w = 34.27 %
TOTAL NORMAL LOAD	36.01 kg
UNIT NORMAL LOAD σ	1.00 kg/cm²

SAMPLE DESCRIPTION BRASS PLATE PRESSED ONTO COMPACTED LAKE EDMONTON CLAY. EXAMINATION OF FAILURE SURFACE SHOWED THAT MOVEMENT OCCURED BETWEEN PLATE & SOIL.

Gs 2.73 S = 90.66 % $\tau_f = 1.09$ kg/cm²

$\gamma = 112.6$ #/ft³, $\gamma_d = 83.84$ #/ft³, $f_{max} = .441$ kg/cm²

P.R. 3-1498

Horizontal Base Movement Turns	Horizontal Base Movement Inches	Proving Ring Dial Reading (.002 mm)	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain	Sample Length cm	Cross-Section Area cm²	Total Shear Force kg	Shear Stress f kg/cm²	f / f_{max}
	0	0	.0800	(ZERO VERTICAL LOAD)						
	0	0	.0783	(1 kg/cm² VERTICAL LOAD)						
	.0010	2	.0783	.0017	.0008	6.00	36.01	0.08	.002	.005
	25	14			14	6.00	36.01	.66	.018	.041
	50	41			18	6.00	36.01	1.92	.053	.120
	80	73			23	6.00	36.01	3.29	.091	.206
	100	90	.0780	.0020	29	6.00	36.01	4.23	.118	.268
	125	111			38	6.00	36.01	5.13	.143	.324
	150	138			41	5.99	35.95	6.33	.176	.399
	175	157	.0779	.0021	51	5.99	35.95	7.18	.200	.453
	200	179			59	5.99	35.95	8.12	.226	.512
	225	202			66	5.99	35.95	9.10	.253	573
	250	228			70	5.99	35.95	10.20	.283	.642
	275	246			81	5.99	35.95	10.95	.304	.689
	300	267			90	5.99	35.95	11.82	.328	.743
	325	287	.0778	.0022	99	5.99	35.95	12.62	.351	.796
	350	302	.0776	.0024	.0112	5.99	35.95	13.25	.368	.835

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 1-B1		TRIAL NO. 1	
LOCATION			
HOLE		DEPTH	
TECHNICIAN MCH		DATE JAN 18, 1964	

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	$f_{max} = 0.441\ kg/cm^2$
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs Dio mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

Horizontal Base Movement		Proving Ring Dial Reading	Vertical Dial Reading	Vertical Strain	Total Horizontal Shear Strain	Sample Length	Cross-Section Area	Total Shear Force	Shear Stress f	$\frac{f}{f_{max}}$
Turns	Inches		In.	In.	In.	cm	cm^2	kg	kg/cm^2	
	.0375	316			.0126	5.99	35.95	13.85	.385	.872
	400	327	.0775	.0025	143	5.99	35.95	14.29	.397	.900
	450	350			174	5.98	35.88	15.22	.424	.961
	500	365			213	5.98	35.88	15.85	.441	1.000
	550	362			265	5.97	35.82	15.72	.438	.993
	600	360			317	5.97	35.82	15.62	.436	.988
	650	356			370	5.96	35.76	15.48	.432	.980
	700	353			422	5.96	35.76	15.35	.429	.972
	750	346			478	5.95	35.70	15.08	.422	.957
	800	342	.0775	.0025	531	5.95	35.70	14.91	.417	.944

SAMPLE F2 - 1-B1 TRIAL 1 SHEET 2 OF 2

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 1-B1 TRIAL NO. 2
LOCATION
HOLE DEPTH
TECHNICIAN MCH DATE JAN. 18, 1964

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION SEE TRIAL 1
WIDTH	FINAL GROSS WEIGHT	TEST RE-RUN BY SLIDING PLATE
LENGTH	WEIGHT OF SPECIMEN	BACK TO ORIGINAL POSITION
INITIAL AREA	VOID RATIO, e	
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 —
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ 1.0 kg/cm^2	Cu $f_{max} = 0.409\ kg/cm^2$

Horizontal Base Movement Turns	Horizontal Base Movement Inches	PR. 3-1498 Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length cm	Cross-Section Area cm^2	Total Shear Force kg	Shear Stress f kg/cm^2	$\frac{f}{f_{max}}$
	0	0	.0775		0	6.00	36.01	0	0	0
	25	27			.0004			1.27	.035	.086
	50	51			.0010			2.38	.066	.161
	75	71			19			3.30	.092	.225
	100	86			32			3.98	.111	.272
	125	99			47			4.59	.128	.313
	150	113			61	5.99	35.95	5.22	.145	.355
	175	132			71			6.08	.169	.413
	200	151			81			6.92	.192	.470
	225	169			92			7.69	.214	.523
	250	188			.0102			8.50	.236	.577
	300	214			131			9.60	.267	.653
	350	252			152	5.98	35.88	11.19	.311	.761
	400	288			173			12.70	.353	.863
	450	322			196			14.10	.392	.958
	500	336			235			14.70	.409	1.000
	550	335			286	5.97	35.82	14.65	.408	.998

SHEET 1 OF 2

SAMPLE F2-1-B1 TRIAL 2 SHEET 1 OF 2

SAMPLE 1- B1 TRIAL NO. 2
LOCATION
HOLE DEPTH
TECHNICIAN McH DATE JAN. 18, 1964

SOIL MECHANICS LABORATORY
DIRECT SHEAR TEST

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

Horizontal Base Movement		Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length cm.	Cross-Section Area cm^2	Total Shear Force kg	Shear Stress f kg/cm^2	$\frac{f}{f_{max}}$
Turns	Inches									
	.0600	334			.0337	5.96	35.76	14.60	.408	.998
	650	331			.389			14.48	.404	.988
	700	329			441			14.38	.402	.982
	750	325			494	5.95	35.70	14.20	.397	.970
	800	323			546			14.11	.395	.965
	.0900	321			647	5.94	35.58	14.06	.394	.963
	.1000	318			750	5.92	35.52	13.92	.391	.956
	.1100	315			852	5.91	35.46	13.80	.389	.952

SAMPLE 1-B2 TRIAL NO. 1
LOCATION
HOLE DEPTH
TECHNICIAN MCH DATE JAN. 18, 1964

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SHEAR BOX:	PLATE	SOIL		
WIDTH	6.000 cm	5.97 cm	INITIAL GROSS WEIGHT	149.61 gms
LENGTH	6.002	6.01	FINAL GROSS WEIGHT	34.51
INITIAL AREA	36.01 cm^2	35.88	WEIGHT OF SPECIMEN	115.10
HEIGHT OF SPECIMEN	1.77 cm		VOID RATIO, e = 1.026 ; w = 34.46 %	
VOLUME OF SPECIMEN	63.51 cm^3		TOTAL NORMAL LOAD	76.02 kg
			UNIT NORMAL LOAD σ =	2.00 kg/cm^2

SAMPLE DESCRIPTION BRASS PLATE B2 PRESSED ONTO COMPACTED LAKE EDMONTON CLAY. FAILURE OCCURRED BETWEEN PLATE & SOIL.

Gs 2.73 $S = 91.69\%$ $\tau_f = 1.09\ kg/cm^2$

$\gamma = 113.1\ \#/ft^3$; $\gamma_d = 84.09\ \#/ft^3$; $f_{max} = 0.692\ kg/cm^2$

Horizontal Base Movement: Turns	Horizontal Base Movement: Inches	P.R. 3-1498 Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length cm	Cross-Section Area cm^2	Total Shear Force kg	Shear Stress f kg/cm^2	f / f_{max}
	0	0	.500	0	(ZERO NORMAL LOAD)					
	0	0	.472	.028	($2 kg/cm^2$ NORMAL LOAD)			0		
	.0025	15			.0014	6.00	36.01	0.71	0.020	0.029
	50	42			17			1.96	.054	.078
	75	69			21			3.21	.089	.129
	100	98			23			4.53	.126	.182
	125	123			28			5.68	.158	.228
	150	146	.471	.029	35			6.68	.186	.269
	175	170			41			7.72	.214	.309
	200	197			45			8.89	.247	.357
	250	247	.470	.030	56	5.99	35.95	11.00	.305	.441
	300	295			68			12.98	.361	.522
	350	324			95			14.15	.393	.568
	400	353			122			15.37	.427	.617
	450	394	.469	.031	140			17.02	.473	.684
	500	436			157	5.98	35.88	18.69	.520	.752
	550	477			174			20.35	.567	.815

SHEET 1 OF 2

SAMPLE F2-1-B2 TRIAL 1 SHEET 1 OF 2

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 1- B2 TRIAL NO. 1
LOCATION
HOLE DEPTH
TECHNICIAN M.C.H. DATE JAN 18, 1964

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	f_{max} = 0.692 kg/cm^2
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

Horizontal Base Movement Turns	Horizontal Base Movement Inches	Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length Cm.	Cross-Section Area cm^2	Total Shear Force Kg.	Shear Stress f kg/cm^2	$\frac{f}{f_{max}}$
	.0600	515			.0194	5.98	35.88	21.90	0.610	.882
	650	547			219	5.97	35.82	23.20	.648	.937
	700	547			269			23.20	.648	.937
	750	570			301			24.10	.673	.974
	800	587			338			24.15	.691	999
	850	587			388	5.96	35.76	24.15	.692	1.000
	900	577			446			24.30	.679	.982
	950	560			509	5.95	35.70	23.70	.663	.959
	1000	550			567	5.94	35.64	23.30	.653	.934
	1050	531	.466	.034	632			22.50	.631	.913
	1200	500			806	5.92	35.52	21.30	.599	.867
			.479	.021 -	(ZERO NORMAL LOAD)					

SHEET 2 OF 2

SAMPLE F2- 1-B2 TRIAL 1 SHEET 2 OF 2

SAMPLE 2-81 TRIAL NO. 1
LOCATION
HOLE DEPTH
TECHNICIAN MCH DATE JAN 18, 1964

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SHEAR BOX:	PLATE	SOIL
WIDTH	6.000 cm	5.97 cm
LENGTH	6.002	5.92
INITIAL AREA	36.01	35.34
HEIGHT OF SPECIMEN	1.77 cm	
VOLUME OF SPECIMEN	62.55 cm^3	

INITIAL GROSS WEIGHT	146.05 gms
FINAL GROSS WEIGHT	33.28
WEIGHT OF SPECIMEN	112.77
VOID RATIO, e = 1.036 ; w = 34.41 %	
TOTAL NORMAL LOAD	72.02 kg
UNIT NORMAL LOAD σ	2.00 kg/cm^2

SAMPLE DESCRIPTION Specimen of Compacted Lake Edmonton Clay Trimmed & Loaded @ 2 kg/cm^2 against Brass Plate for 40 min in Shear Box. At Failure Some Soil Adhered to Plate. No Slickeslides

Gs 2.73 S = 90.68 % T_f = 1.09 kg/cm^2

γ = 112.5#/ft³, γ_d = 83.69#/ft³, f_{max} = .995 kg/cm^2

Horizontal Base Movement: Time (Min.)	Horizontal Base Movement: Inches	PR 3-1498 Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length Cm.	Cross-Section Area cm^2	Total Shear Force kg	Shear Stress f kg/cm^2	$\frac{f}{f_{max}}$
0	0	0	.500	0	(Zero Normal Load)					
40		0	.451	.049	(2 kg/cm^2 Normal Load)					
00 = 40	25	30			.0001	6.00	36.01	1.41	0.039	.039
	50	58			4			2.70	.075	.075
	75	84			9			3.89	.108	.108
	100	108			.0015			4.98	.138	.139
	125	134			19			6.16	.171	.172
	150	157			26			7.16	.198	.199
	175	183			31			8.27	.229	.230
0.7	200	207			37			9.29	.258	.259
	225	229			45			10.24	.284	.285
	250	251			52	5.99	35.95	11.15	.309	.311
	275	268			64			11.84	.329	.331
1.3	300	285			76			12.57	.349	.351
	350	325			.0100			14.20	.394	.396
1.7	400	360			117			15.65	.435	.437
	450	407			130			17.53	.487	.489

SAMPLE F2-2B1 TRIAL 1 SHEET 1 OF 2

DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 2-B1	TRIAL NO. 1
LOCATION	
HOLE	DEPTH
TECHNICIAN MCH	DATE JAN 18, 1964

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	$f_{max} = 0.995$ kg/cm²
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

Horizontal Base Movement: Time (Min)	Horizontal Base Movement: Inches	PR. 3-1498 Proving Ring Dial Reading .002 mm	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Strain In.	Sample Length Cm.	Cross-Section Area Cm²	Total Shear Force kg.	Shear Stress f kg/cm²	$\frac{f}{f_{max}}$
2.1 min	.0500	.452	.4505	0495	.0144	5.99	35.95	19.35	0.538	.540
	550	491			163	5.98	35.88	20.70	.576	.578
	600	539			176			22.80	.635	.638
	650	570			201			24.10	.671	.674
	700	603	.450	.050	225			25.40	.707	.710
	750	635			250			26.75	.745	.748
	800	668			274	5.97	35.82	28.00	.781	.784
	850	-	.449	.052	-			-		
	900	759			302			31.60	.882	.887
	950	784			334			32.60	.910	.914
	1000	815			358	5.96	35.76	33.90	.947	.952
	1100	858			424			35.60	.995	1.000
	1200	640	.448	.052	696	5.93	35.58	26.90	.756	.760
			.462	.038 → (ZERO NORMAL LOAD)						

SHEET 2 OF 2

SAMPLE F2-2B1 TRIAL 1 SHEET 1 OF 2

SAMPLE F2-1-DS TRIAL 1 SHEET 1 OF 3

SOIL MECHANICS LABORATORY
DIRECT SHEAR TEST

SAMPLE 1- DS	TRIAL NO. 1
LOCATION	
HOLE	DEPTH
TECHNICIAN MCH	DATE JAN 18, 1964

SHEAR BOX:		INITIAL GROSS WEIGHT	224.87 gms	SAMPLE DESCRIPTION COMPACTED LAKE EDMONTON CLAY.
WIDTH	5.94 cm	FINAL GROSS WEIGHT	33.00	SOIL SHEARED WITH ~ .05 mm CLEARANCE BETWEEN
LENGTH	6.002 cm (2.363")	WEIGHT OF SPECIMEN	191.87 gms.	TOP & BOTTOM PLATE. SKETCH →
INITIAL AREA	35.65 cm²	VOID RATIO, e = 1.057 ; w = 34.24		
HEIGHT OF SPECIMEN	3.02 cm	TOTAL NORMAL LOAD	36.01 kg	Gs 2.73 ~~Dro~~ S = 88.43 % ~~mm~~
VOLUME OF SPECIMEN	107.66 cm³	UNIT NORMAL LOAD σ =	1.01 kg/cm²	~~eo~~ γ = 111.20 #/ft³, γ_d = 82.83 #/ft³

Horizontal Base Movement: Horizontal Shear Strain In.	Horizontal Base Movement: Inches	Proving Ring Dial Reading	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Unit Strain %	Sample Length Cm	Cross-Section Area cm²	Total Shear Force kg.	Shear Stress τ kg/cm²	$\frac{\tau}{\cdot}$
.0	0	0	.400	0	ZERO NORMAL LOAD					
	0	0	.379	.021	1.01 kg/cm² "	"				
.0009	.0025	20			.04	6.00	35.65	.94	.03	
22	50	35			.09	6.00	35.65	1.64	.05	
36	75	49			.15	6.00	35.65	2.29	.06	
50	100	64	.378	.022	.21	6.00	35.65	2.98	.08	
63	125	79			.27	6.00	35.65	3.67	.10	
75	150	95			.32	5.99	35.58	4.40	.12	
87	175	112			.37	5.99	35.58	5.18	.15	
99	200	128			.42	5.99	35.58	5.89	.17	
.0119	250	166			.50	5.99	35.58	7.55	.21	
.0141	300	202			.60	5.99	35.58	9.09	.26	
173	350	237			.73	5.99	35.58	10.58	.30	
193	400	263			.82	5.98	35.52	11.65	.33	
219	450	293			.93	5.98	35.52	12.90	.36	
250	500	317	.377	.023	1.06	5.98	35.52	13.90	.39	
279	550	344			1.18	5.97	35.46	15.00	.42	

SOIL MECHANICS LABORATORY
DIRECT SHEAR TEST

SAMPLE 1- DS	TRIAL NO. 1
LOCATION	
HOLE	DEPTH
TECHNICIAN	DATE JAN 18, 1964

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

Horizontal Base Movement		Proving Ring Dial Reading	Vertical Dial Reading	Vertical Strain	Total Horizontal Shear Unit Strain %	Sample Length	Cross-Section Area	Total Shear Force	Shear Stress τ	$\frac{\tau}{\sigma}$
Turns	Inches		In.	In.		cm	cm^2	kg.	kg/cm^2	
.0309	.0600	369	.377	.023	1.31	5.97	35.46	16.00	0.451	
336	650	398			1.42	5.97	35.46	17.20	.485	
366	700	425			1.55	5.97	35.46	18.30	.516	
381	750	456			1.62	5.96	35.40	19.50	.550	
413	800	481			1.75	5.96	35.40	20.50	.579	
444	850	515			1.88	5.96	35.34	21.90	.619	
483	900	529			2.04	5.95	35.34	22.40	.633	
508	950	560			2.15	5.95	35.34	23.70	.670	
539	.1000	585			2.28	5.95	35.34	24.70	.698	
605	1100	629			2.56	5.94	35.28	26.50	.751	
676	1200	666			2.86	5.94	35.28	28.00	.793	
750	1300	698			3.17	5.93	35.22	29.20	.829	
828	1400	727			3.50	5.92	35.16	30.40	.864	
911	1500	748			3.86	5.91	35.11	31.90	.908	
998	1600	765			4.22	5.90	35.05	32.60	.930	
.1080	1700	788			4.57	5.89	34.99	33.60	.961	
1168	1800	802			4.94	5.89	34.99	34.10	.974	

SHEET 2 OF 3

SAMPLE F2-1-DS TRIAL 1 SHEET 2 OF 3

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

SAMPLE 1- DS TRIAL NO. 1
LOCATION
HOLE DEPTH
TECHNICIAN DATE JAN 18, 1964

SHEAR BOX:	INITIAL GROSS WEIGHT	SAMPLE DESCRIPTION
WIDTH	FINAL GROSS WEIGHT	
LENGTH	WEIGHT OF SPECIMEN	
INITIAL AREA	VOID RATIO, e	
HEIGHT OF SPECIMEN	TOTAL NORMAL LOAD	Gs D10 mm
VOLUME OF SPECIMEN	UNIT NORMAL LOAD σ	Cu

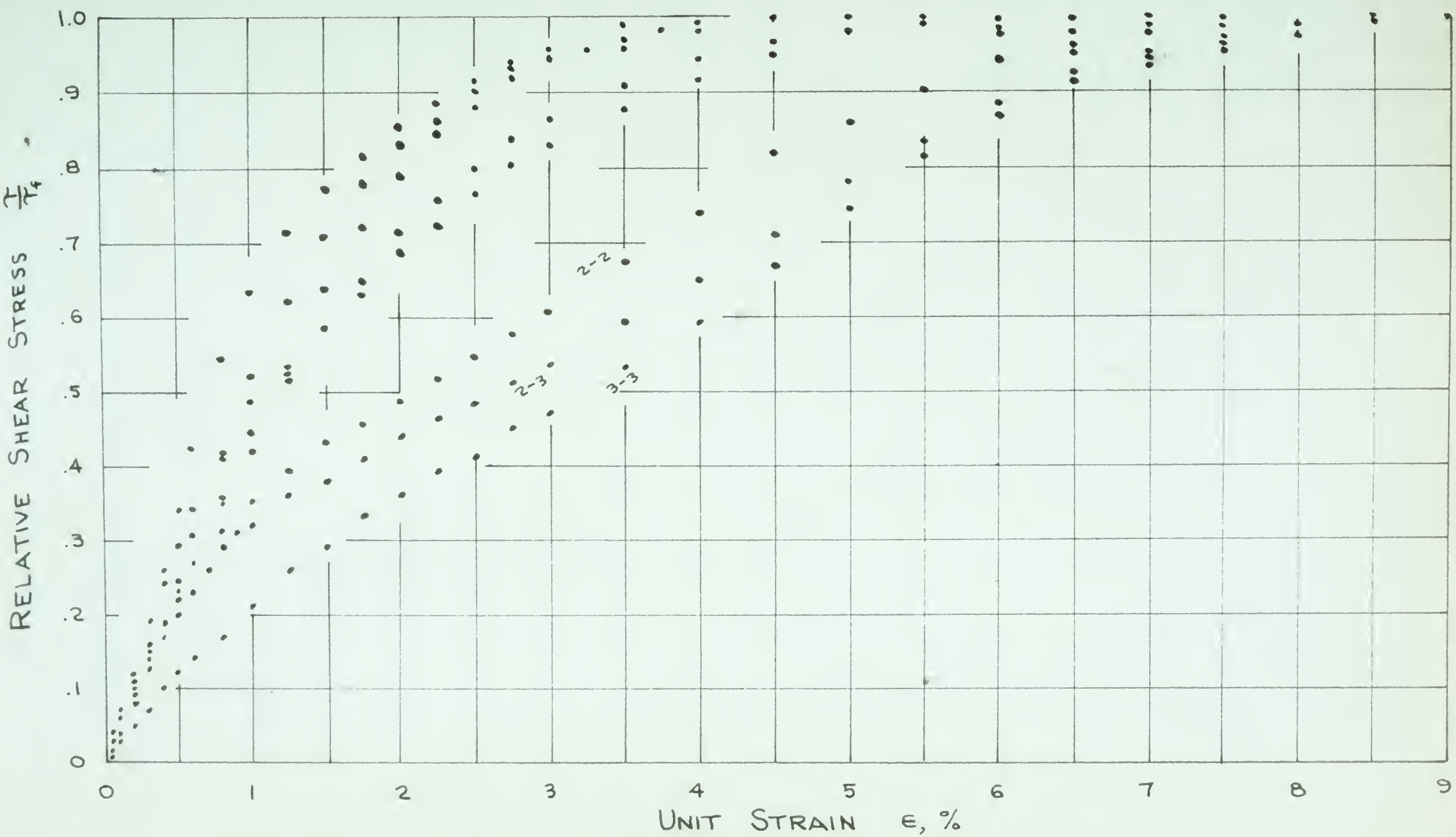
Horizontal Base Movement Turns	Horizontal Base Movement Inches	Proving Ring Dial Reading	Vertical Dial Reading In.	Vertical Strain In.	Total Horizontal Shear Unit Strain %	Sample Length Cm	Cross-Section Area Cm^2	Total Shear Force kg.	Shear Stress τ kg/cm^2	$\frac{\tau}{\sigma}$
.1261	.1900	812			5.33	5.88	34.93	34.6	0.991	
.1364	2000	809			5.77	5.87	34.87	34.4	0.987	
.1455	2100	819			6.16	5.86	34.81	34.8	1.000	
1564	2200	808			6.62	5.85	34.75	34.4	0.990	
1686	2300	780			7.14	5.83	34.63	33.3	0.962	
1782	2400	785			7.54	5.82	34.57	33.7	0.974	
1906	2500	755			8.07	5.81	34.51	32.3	0.936	
1997	2600	765	.386	.014	8.45	5.80	34.45	32.7	0.949	
			.393	.007	(ZERO NORMAL LOAD.)					

SAMPLE F2-1-DS TRIAL 1 SHEET 3 OF 3

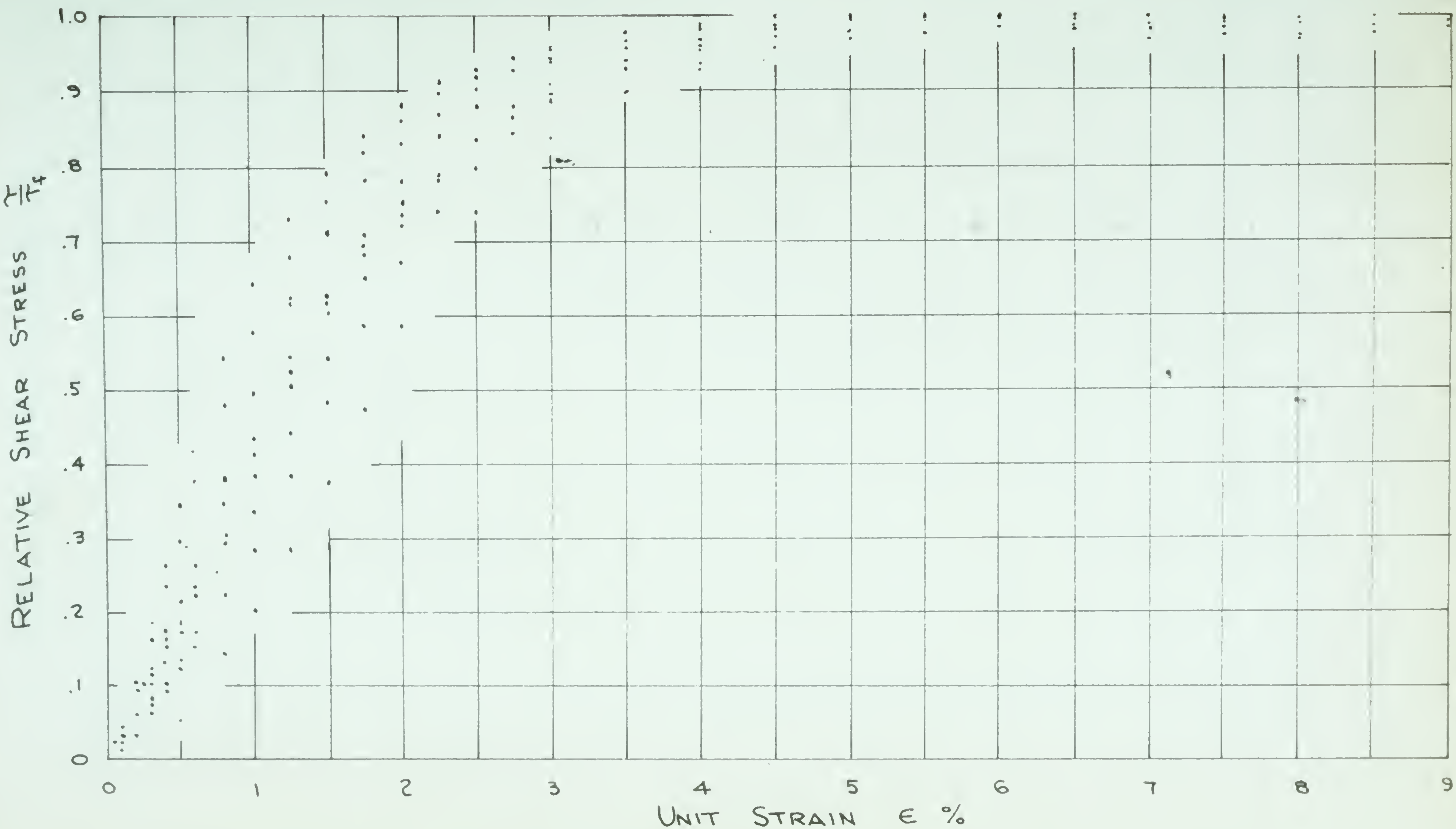
APPENDIX J

STRESS STRAIN CURVES FOR SHAFT AND BASE SOILS

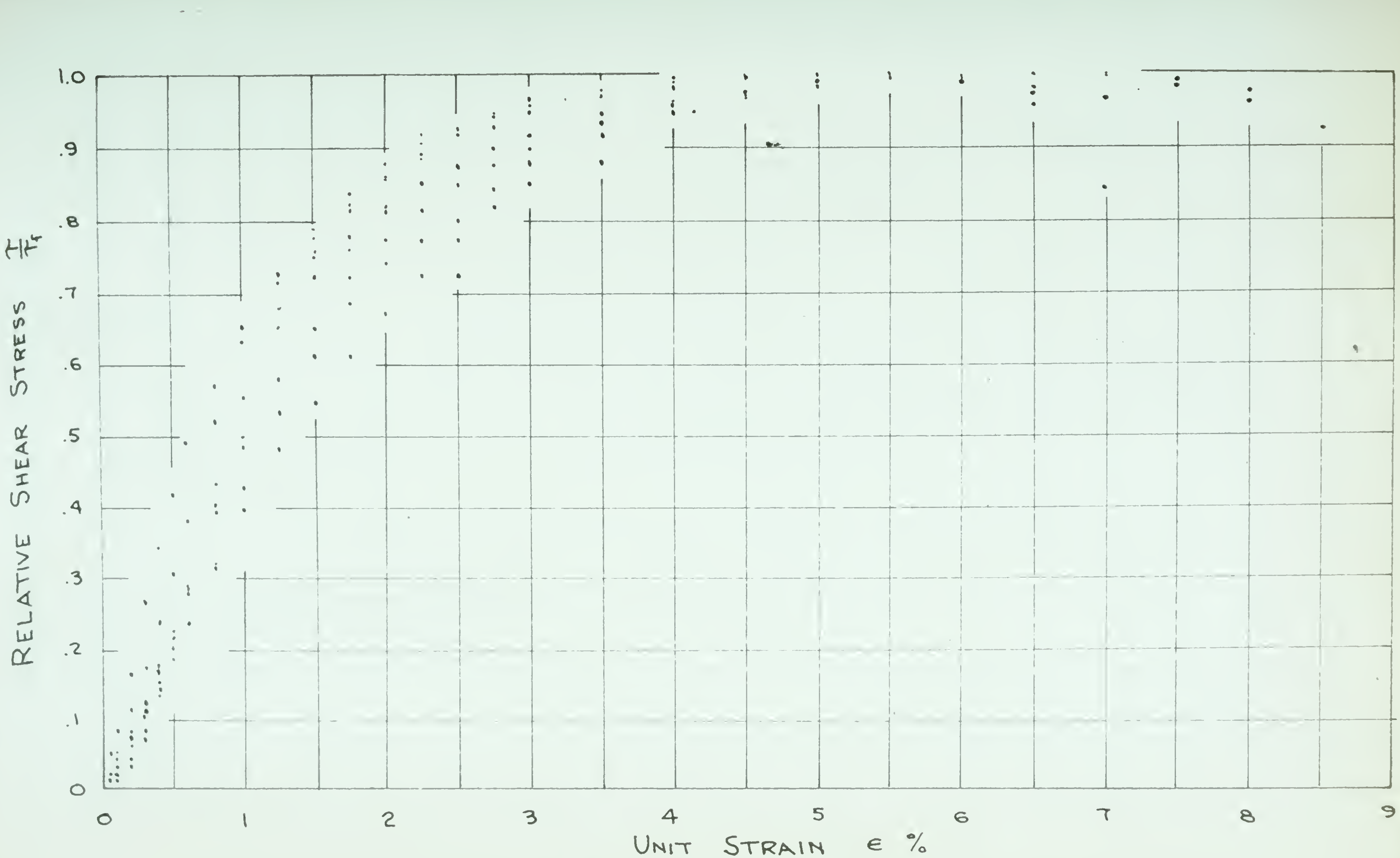
(Tests A-1 to A-4)



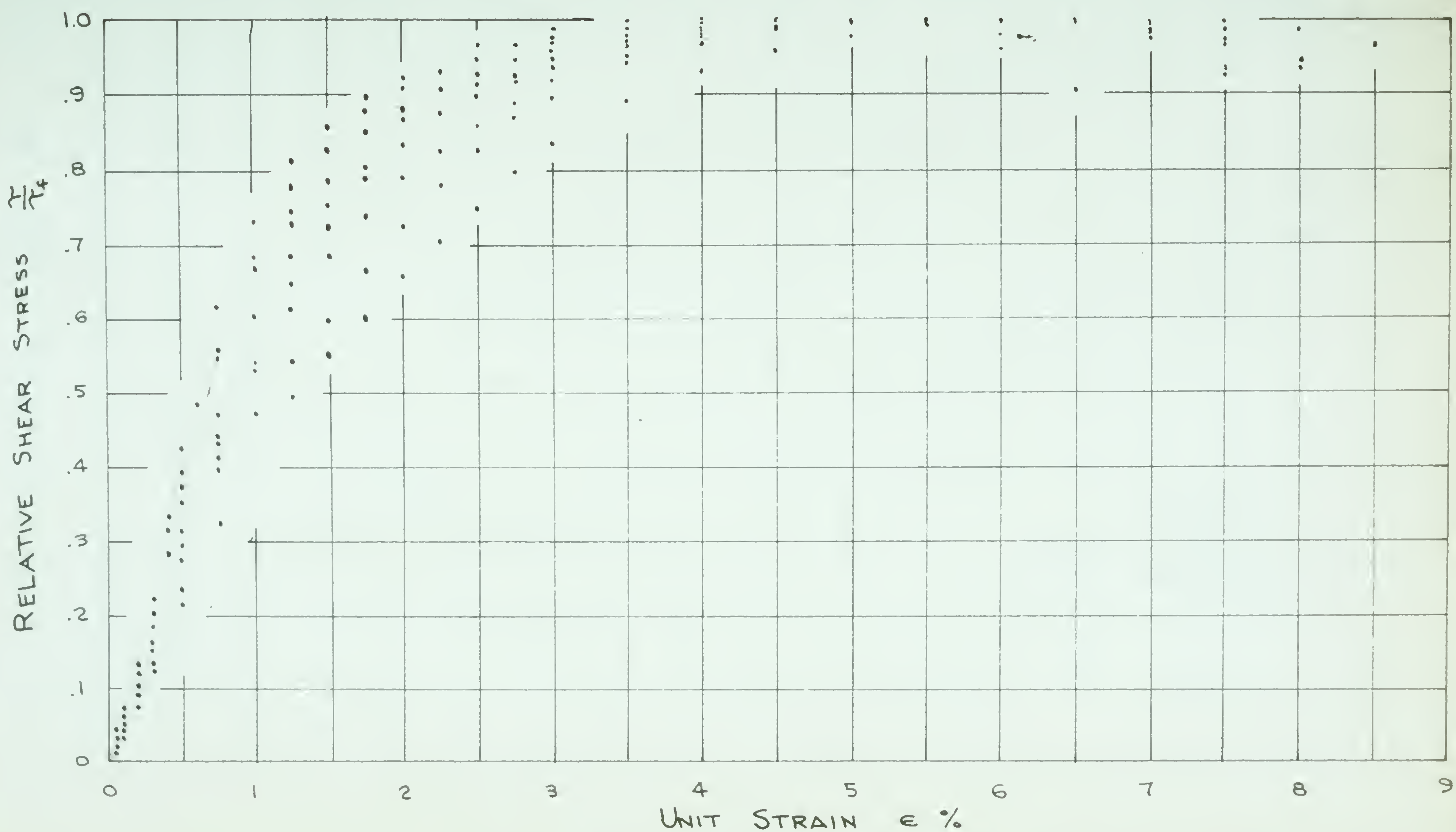
TEST A-1 SHAFT SOIL



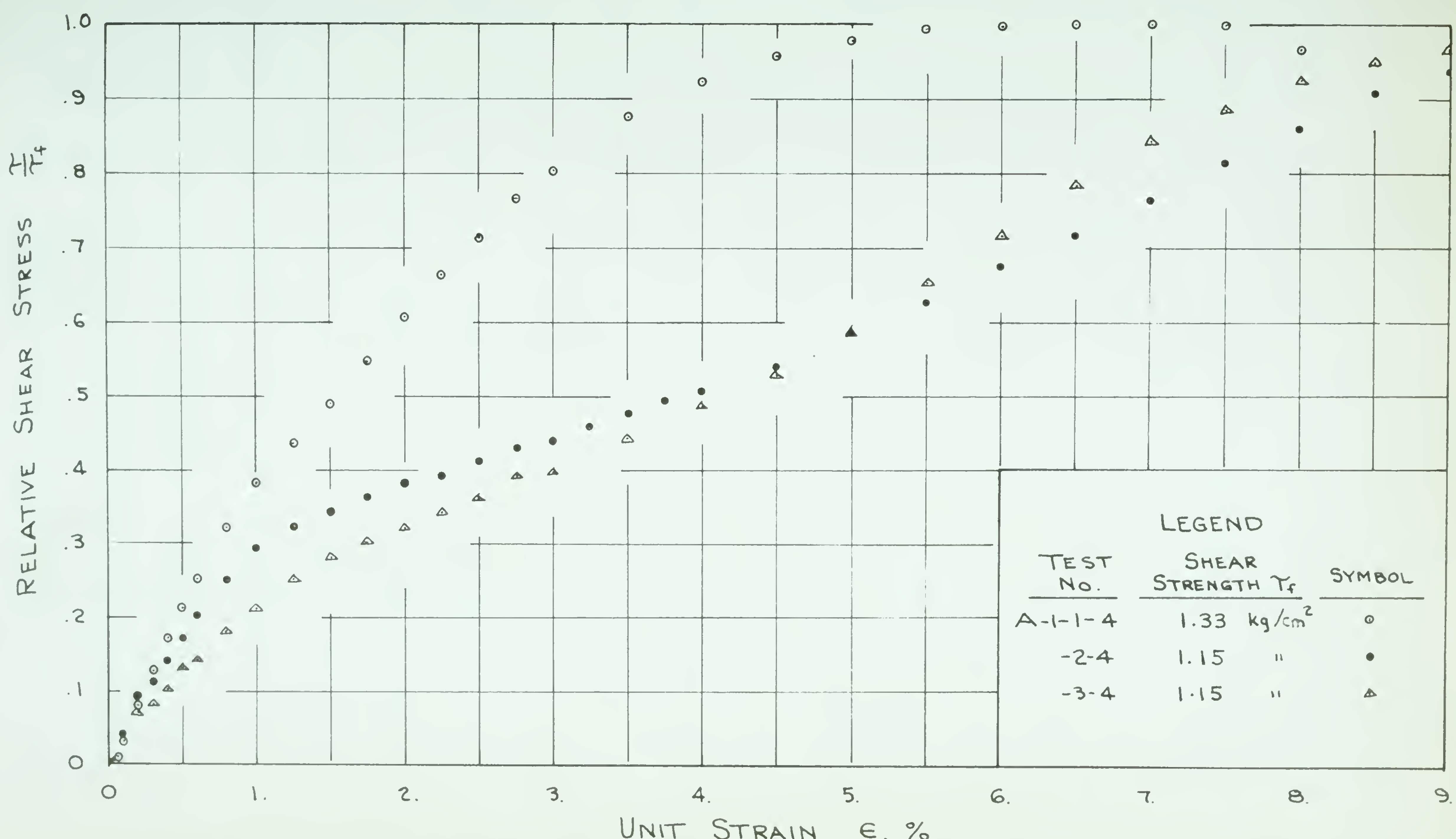
TEST A-2 SHAFT SOIL



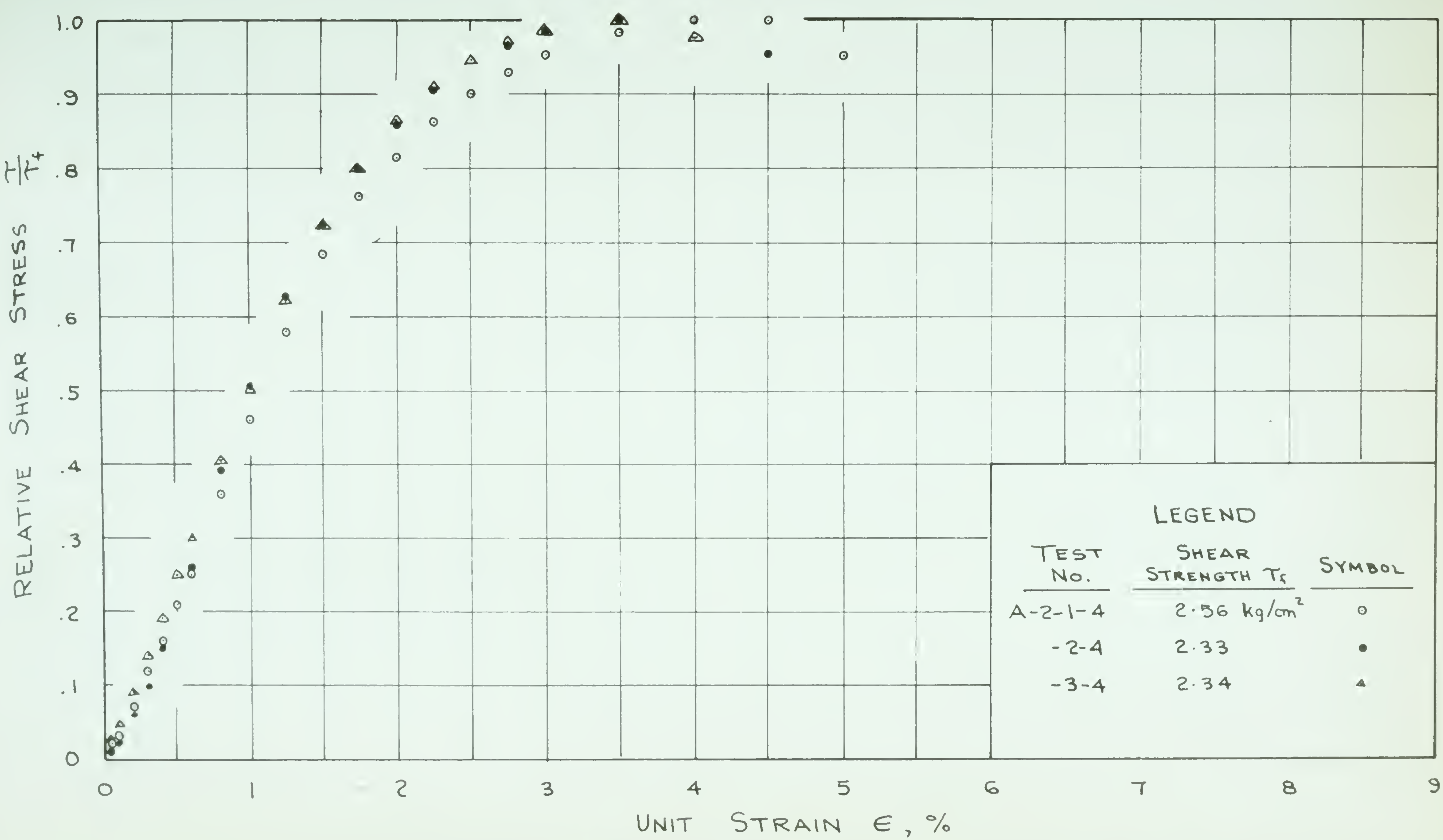
TEST A3 SHAFT SOIL



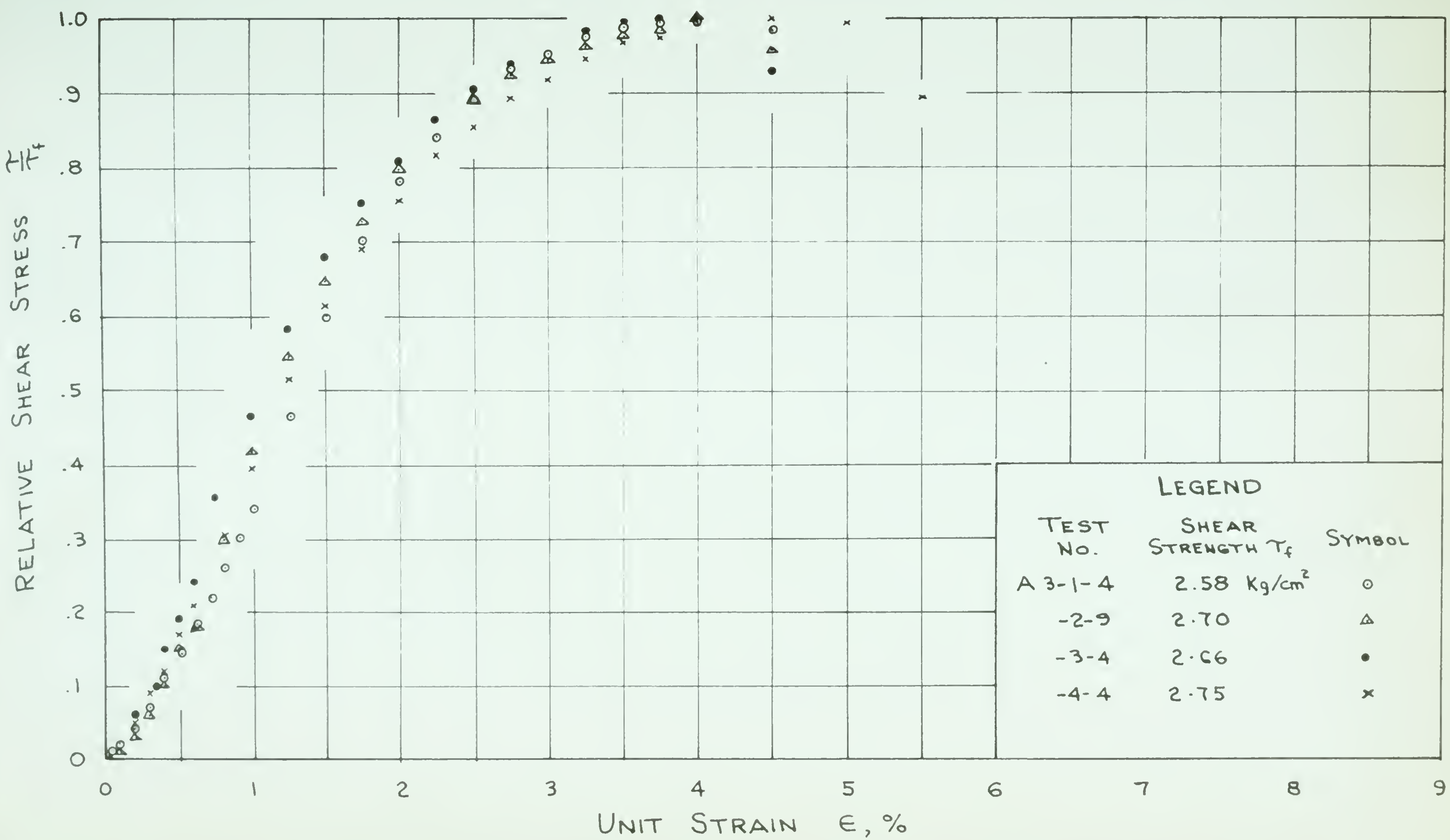
Test A-4 Shaft Soil



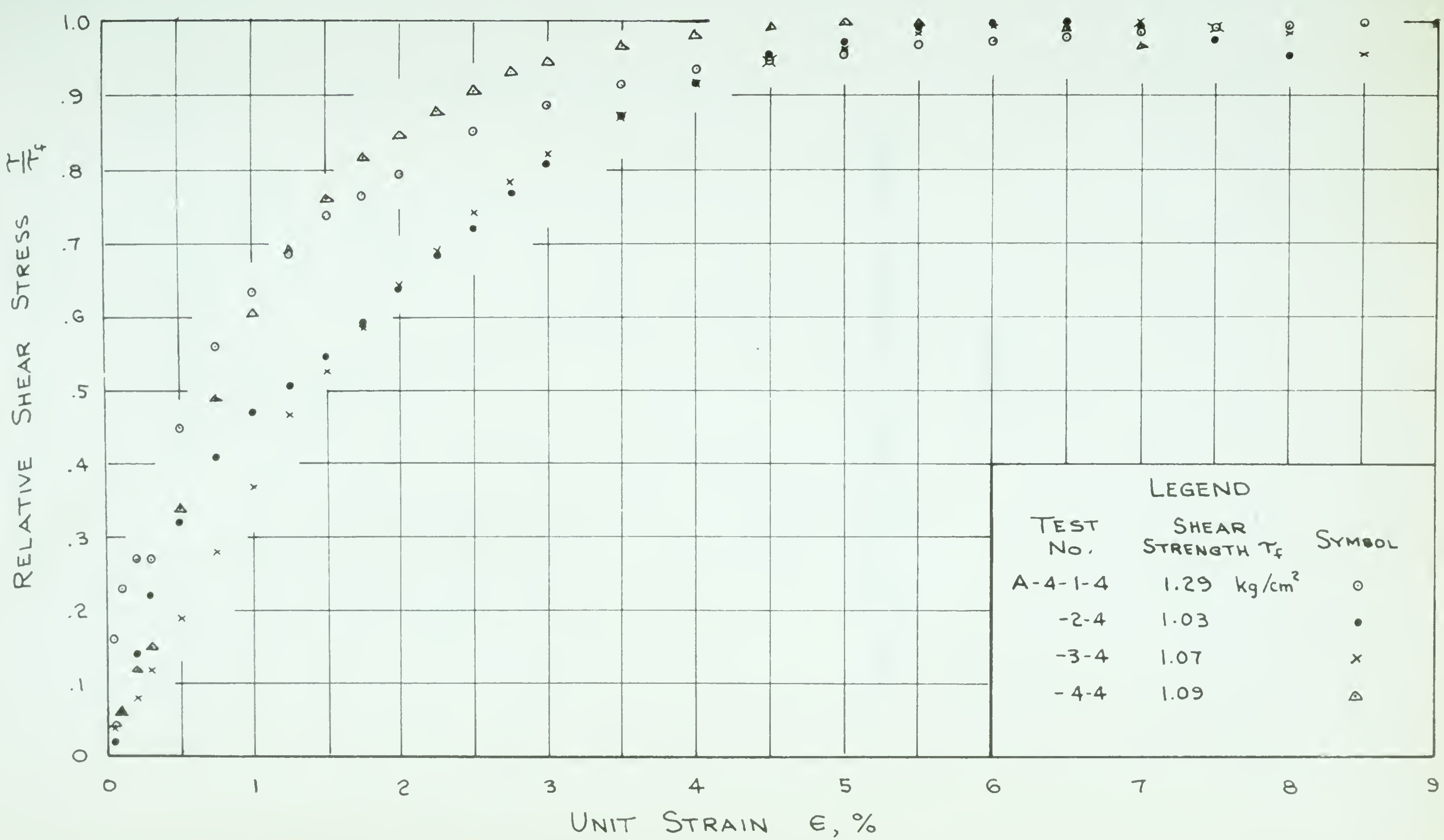
Test A1 Base Soil



TEST A2 BASE SOIL



TEST A 3 BASE SOIL



Test A4 Base Soil

APPENDIX K

COMPUTER PROGRAM & OUTPUT FOR PILE TESTS

COMPUTER PROGRAM AND OUTPUT FOR PILE TESTS

The programs for both the calibration and pile tests are in the standard Fortran format required by the IBM 1620 computer. The calculations are simple ones with operations as given by the following symbols:

*	Multiply
**	raise to power....
/	divide
-	subtract
+	add
=	equals

The abbreviated symbols required for the program have the following meanings:

NBR	-Test Number (ie 111= A-1a)
BASE	-Base diameter (inches)
DPTH	-Base depth (inches)
SHFT	-Shaft length (inches)
QMAX	-Maximum pile load (kgms)
ARP	-Area of pile shaft materials (cm^2)
THR	-Thickness ratio of pile shaft (2t/b)
EMP	-Modulus of elasticity of shaft material ($psix10^{-6}$)
PRA	-Poisson's ratio for shaft material
DEL	-Length of shaft (inches) for which gauge gives average vertical strain
CONA to CONF	-calculation constants for shaft loads and stresses
M	-number of pile loads
N	-number of gauge levels (6)
QUT	-pile load (kgms)
PENT	-Penetration of pile cap (inches x 10^{-4})
QBASE	-Base load (kgms)

GDEP	-Gauge depth (inches) from soil surface (H)
SVL	-strain reading for vertical strain gauge, left side of pile (inches x 10^{-4})
SVR	-as above for SVL except right side of pile
SHL	-strain readings for horizontal strain gauge, left side of pile (inches x 10^{-4})
SHR	-strain reading for horizontal strain gauge, right side of pile (inches x 10^{-4})
QH	-pile load at depth H (kgms)
QHR	-ratio pile load to maximum pile load
P	-lateral pressure on side of pile (kg/cm^2)
DR	-ratio of gauge depth to shaft length
DEF	-deformation of pile over length of shaft given by vertical strain at gauge depth H
SMFD=(SD)	-summation of deformation of pile at depth H (inches x 10^{-4})
PED	-penetration of pile at depth H (inches x 10^{-4})

The output message "ERROR" which appears at the end of some of the output sheets is simply a routine diagnostic statement indicating that the computer was unable to read more data cards and thus continue the program for the next test.

```
..I 920130 MODEL PILE  ROUTINE A  M.C. HARRIS
..LOAD FORGO CLOCK 600
 1     READ 2,NBR,BASE,DPTH,SHFT,QMAX
 2     FORMAT(1X,I4,4F10.4)
       READ 3,ARP,THR
 3     FORMAT(2F10.4)
       READ 4,EMP,PRA
 4     FORMAT(2F10.4)
       DIMENSION DEL(6)
 60    READ 61,(DEL(J),J=1,6)
 61    FORMAT(6F10.4)
 5     CONA=EMP/(28.446*(1.-PRA**2))
       CONB=CONA*PRA
       CONC=CONA*ARP
       COND=CONB*ARP
       CONE=CONA*THR
       CONF=CONB*THR
 6     PUNCH 7,NBR,BASE,QMAX
 7     FORMAT(9H1TEST NO.,I4,2X,9HBASE DIA.,F5.2,2X,8HMAX LOAD,F7.2)
       PUNCH 8,DPTH,SHFT
 8     FORMAT(1X,6HDEPTH=,F7.2,5X,13HSHAFT LENGTH=,F7.2)
       PUNCH 9,ARP,THR
 9     FORMAT(1X,10HPILE AREA=,F10.4,4X,12HTHICK RATIO=,F10.4)
       PUNCH 10,EMP,PRA
 10    FORMAT(1X,15HMOD ELAST PILE=,F7.3,2X,14HPOISSON RATIO=,F7.4//)
       PUNCH 11,CONA,CONB,CONC
 11    FORMAT(1X,3HKA=,F10.4,2X,3HKB=,F10.4,2X,3HKC=,F10.4)
       PUNCH 12,COND,CONE,CONF
 12    FORMAT(1X,3HKD=,F10.4,2X,3HKE=,F10.4,2X,3HKF=,F10.4)
 13    READ 14,M,N
 14    FORMAT(1X,2I4)
 15    PUNCH 16,M,N
 16    FORMAT(//1X,15HNO. PILE LOADS=,I4,5X,11HNO. DEPTHS=,I4)
       DO 45 I=1,M
       SMDF=0.
 17    READ 18,QUT,PENT,QBASE
 18    FORMAT(3F10.4)
 19    PUNCH 20,QUT,PENT
 20    FORMAT(//1X,10HPILE LOAD=,F7.2,4X,12HPENETRATION=,F8.1)
 90    PUNCH 91,QBASE
 91    FORMAT(1X,10HBASE LOAD=,F7.2,2X,5HKGMS./)
 21    PUNCH 22
 22    FORMAT(3X,4HGDEP,7X,2HDR,8X,1HP,8X,2HQH,8X,3HDEF,7X,2HSD,7X,3HPED)
       DO 45 J=1,N
 23    READ 24,GDEP,SVL,SVR,SHL,SHR
 24    FORMAT(5F10.4)
 25    QH=CONC*(SVL+SVR)+COND*(SHL+SHR)
 26    P=CONE*(SHL+SHR)+CONF*(SVL+SVR)
       DR=GDEP/DPTH
 63    DEF=(SVL+SVR)/200.*DEL(J)
       SMDF=SMDF+DEF
 64    PED=PENT-SMDF
 27    PUNCH 28,GDEP,DR,P,QH,DEF,SMDF,PED
 28    FORMAT(F8.2,F9.3,F10.3,F10.2,F10.3,2F10.2)
 45    CONTINUE
       GO TO 1
       END
```

```
..I 920130 MODEL PILE  ROUTINE B  M.C. HARRIS
..LOAD FORGO CLOCK 600
 76    READ 77,NBR,BASE,DPTH
 77    FORMAT(1X,I4,2F10.4)
       READ 3,ARP,THR
 3     FORMAT(2F10.4)
       READ 4,EMP,PRA
 4     FORMAT(2F10.4)
 5     CONA=EMP/(28.446*(1.-PRA**2))
       CONB=CONA*PRA
       CONC=CONA*ARP
       COND=CONB*ARP
       CONE=CONA*THR
       CONF=CONB*THR
 6     PUNCH 7,NBR,BASE
 7     FORMAT(9H1TEST NO.,I4,2X,9HBASE DIA.,F5.2)
 78    PUNCH 79,DPTH
 79    FORMAT(7H DEPTH=,F7.3,6HINCHES)
       PUNCH 9,ARP,THR
 9     FORMAT(1X,10HPILE AREA=,F10.4,4X,12HTHICK RATIO=,F10.4)
       PUNCH 10,EMP,PRA
 10    FORMAT(1X,15HMOD ELAST PILE=,F10.4,4X,14HPOISSON RATIO=,F10.4)
       PUNCH 11,CONA,CONB,CONC
 11    FORMAT(1X,3HKA=,F10.4,2X,3HKB=,F10.4,2X,3HKC=,F10.4)
       PUNCH 12,COND,CONE,CONF
 12    FORMAT(1X,3HKD=,F10.4,2X,3HKE=,F10.4,2X,3HKF=,F10.4)
 13    READ 14,M,N
 14    FORMAT(1X,2I4)
 15    PUNCH 16,M,N
 16    FORMAT(//1X,15HNO. PILE LOADS=,I4,5X,11HNO. DEPTHS=,I4)
       DO 45 I=1,M
 72    READ 73,QUT
 73    FORMAT(F10.4)
 74    PUNCH 75,QUT
 75    FORMAT(//1X,10HPILE LOAD=,F7.2,5HKGMS.)
 70    PUNCH 71
 71    FORMAT(5X,4HGDEP,9X,1HP,8X,2HQH,9X,2HQR,4X,11H(SVL+SVR)/2)
       DO 45 J=1,N
 23    READ 24,GDEP,SVL,SVR,SHL,SHR
 24    FORMAT(5F10.4)
 25    QH=CONC*(SVL+SVR)+COND*(SHL+SHR)
 80    QR=QH/QUT
 26    P=CONE*(SHL+SHR)+CONF*(SVL+SVR)
 50    SAVG=(SVL+SVR)/2.
 81    PUNCH 82,GDEP,P,QH,QR,SAVG
 82    FORMAT(2F10.3,F10.2,F10.3,F10.2)
 45    CONTINUE
       GO TO 76
       END
```

```
..I 920130 MODEL PILE  ROUTINE C  M.C. HARRIS  PLEASE SAVE OUTPUT CARDS
..LOAD FORGO CLOCK 600
 1     READ 2,NBR,BASE,DPTH,SHFT,QMAX
 2     FORMAT(1X,I4,4F10.4)
       READ 3,ARP,THR
 3     FORMAT(2F10.4)
       READ 4,EMP,PRA
 4     FORMAT(2F10.4)
       DIMENSION DEL(6)
 60    READ 61,(DEL(J),J=1,6)
 61    FORMAT(6F10.4)
 5     CONA=EMP/(28.446*(1.-PRA**2))
       CONB=CONA*PRA
       CONC=CONA*ARP
       COND=CONB*ARP
       CONE=CONA*THR
       CONF=CONB*THR
 6     PUNCH 7,NBR,BASE,QMAX
 7     FORMAT(9H1TEST NO.,I4,2X,9HBASE DIA.,F5.2,2X,8HMAX LOAD,F7.2)
       PUNCH 8,DPTH,SHFT
 8     FORMAT(1X,6HDEPTH=,F7.2,5X,13HSHAFT LENGTH=,F7.2)
       PUNCH 9,ARP,THR
 9     FORMAT(1X,10HPILE AREA=,F10.4,4X,12HTHICK RATIO=,F10.4)
       PUNCH 10,EMP,PRA
 10    FORMAT(1X,15HMOD ELAST PILE=,F7.3,2X,14HPOISSON RATIO=,F7.4//)
       PUNCH 11,CONA,CONB,CONC
 11    FORMAT(1X,3HKA=,F10.4,2X,3HKB=,F10.4,2X,3HKC=,F10.4)
       PUNCH 12,COND,CONE,CONF
 12    FORMAT(1X,3HKD=,F10.4,2X,3HKE=,F10.4,2X,3HKF=,F10.4)
 13    READ 14,M,N
 14    FORMAT(1X,2I4)
 15    PUNCH 16,M,N
 16    FORMAT(//1X,15HNO. PILE LOADS=,I4,5X,11HNO. DEPTHS=,I4)
       DO 45 I=1,M
       SMDF=0.
 17    READ 18,QUT,PENT,QBASE
 18    FORMAT(3F10.4)
 19    PUNCH 20,QUT,PENT
 20    FORMAT(//1X,10HPILE LOAD=,F7.2,4X,12HPENETRATION=,F8.1)
 90    PUNCH 91,QBASE
 91    FORMAT(1X,10HBASE LOAD=,F7.2,2X,5HKGMS./)
       QUTR=QUT/QMAX
 92    PUNCH 93,QUTR
 93    FORMAT(1X,16HPILE LOAD RATIO=,F7.2)
 21    PUNCH 22
 22    FORMAT(3X,4HGDEP,7X,2HDR,8X,2HQH,7X,3HQHR)
       DO 45 J=1,N
 23    READ 24,GDEP,SVL,SVR,SHL,SHR
 24    FORMAT(5F10.4)
 25    QH=CONC*(SVL+SVR)+COND*(SHL+SHR)
       QHR=QH/QMAX
 26    P=CONE*(SHL+SHR)+CONF*(SVL+SVR)
       DR=GDEP/DPTH
 63    DEF=(SVL+SVR)/200.*DEL(J)
       SMDF=SMDF+DEF
 64    PED=PENT-SMDF
```

```
97    PUNCH 98,GDEP,DR,QH,QHR
98    FORMAT(F8.2,F9.3,F10.3,F10.3)
45    CONTINUE
      GO TO 1
      END
```

```
..I 920130 MODEL PILE  ROUTINE A  M.C. HARRIS  PLEASE SAVE OUTPUT CARDS
..LOAD FORGO CLOCK 600
 TEST NO. 111  BASE DIA. 2.00  MAX LOAD 245.00
 DEPTH=  10.88      SHAFT LENGTH=   9.85
 PILE AREA=    1.0700    THICK RATIO=      .1048
 MOD ELAST PILE= 15.380  POISSON RATIO=  .3100

 KA=      .5982  KB=       .1854  KC=      .6400
 KD=      .1984  KE=       .0627  KF=      .0194

 NO. PILE LOADS=  22      NO. DEPTHS=   6
```

PILE LOAD= 20.00   PENETRATION=   7.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-.887	17.44	.613	.61	6.39
-.94	-.086	.150	23.62	.350	.96	6.04
1.06	.097	.053	20.42	.394	1.36	5.64
3.56	.327	.367	21.41	.438	1.79	5.21
6.06	.557	.486	16.00	.313	2.11	4.89
8.56	.787	.291	9.60	.155	2.26	4.74

PILE LOAD= 40.00   PENETRATION=   12.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.204	44.03	1.313	1.31	10.69
-.94	-.086	.517	45.03	.656	1.97	10.03
1.06	.097	.204	44.03	.844	2.81	9.19
3.56	.327	.009	37.63	.813	3.63	8.38
6.06	.557	.031	29.03	.625	4.25	7.75
8.56	.787	.583	19.20	.309	4.56	7.44

PILE LOAD= 54.20   PENETRATION=   17.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.279	55.84	1.663	1.66	15.34
-.94	-.086	.160	61.25	.919	2.58	14.42
1.06	.097	.160	61.25	1.181	3.76	13.24
3.56	.327	.592	56.83	1.188	4.95	12.05
6.06	.557	.517	45.03	.938	5.89	11.11
8.56	.787	.874	28.80	.464	6.35	10.65

PILE LOAD= 80.00   PENETRATION=   26.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.310	84.87	2.538	2.54	23.46
-.94	-.086	.818	92.26	1.356	3.89	22.11
1.06	.097	.602	94.47	1.800	5.69	20.31
3.56	.327	.840	83.65	1.750	7.44	18.56
6.06	.557	.041	66.66	1.438	8.88	17.12
8.56	.787	.928	49.22	.824	9.71	16.29

PILE LOAD= 100.00   PENETRATION=   36.0

GDEP	DR	P	QH	DEF	SD	PED

-2.44	-.224	.774	109.48	3.238	3.24	32.76
-.94	-.086	.774	109.48	1.619	4.86	31.14
1.06	.097	.655	114.88	2.194	7.05	28.95
3.56	.327	.893	104.07	2.188	9.24	26.76
6.06	.557	.505	91.27	1.938	11.18	24.83
8.56	.787	1.197	67.43	1.133	12.31	23.69

PILE LOAD= 120.00 PENETRATION= 59.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.827	129.89	3.850	3.85	55.15
-.94	-.086	.514	128.90	1.925	5.78	53.23
1.06	.097	1.022	136.29	2.588	8.36	50.64
3.56	.327	1.044	127.69	2.688	11.05	47.95
6.06	.557	.244	110.69	2.375	13.43	45.58
8.56	.787	.310	84.87	1.494	14.92	44.08

PILE LOAD= 140.00 PENETRATION= 111.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.784	147.11	4.375	4.38	106.63
-.94	-.086	.157	145.13	2.188	6.56	104.44
1.06	.097	.254	148.33	2.869	9.43	101.57
3.56	.327	1.097	148.10	3.125	12.56	98.44
6.06	.557	-.113	126.92	2.750	15.31	95.69
8.56	.787	1.012	98.66	1.700	17.01	93.99

PILE LOAD= 160.00 PENETRATION= 189.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.210	165.54	4.988	4.99	184.01
-.94	-.086	-.201	161.35	2.450	7.44	181.56
1.06	.097	.091	170.95	3.319	10.76	178.24
3.56	.327	.621	169.74	3.625	14.38	174.62
6.06	.557	.254	148.33	3.188	17.57	171.43
8.56	.787	.677	106.28	1.854	19.42	169.58

PILE LOAD= 180.00 PENETRATION= 326.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.771	193.35	5.775	5.78	320.23
-.94	-.086	.458	192.36	2.888	8.66	317.34
1.06	.097	.144	191.37	3.713	12.38	313.63
3.56	.327	.577	186.95	4.000	16.38	309.63
6.06	.557	.815	176.14	3.750	20.13	305.88
8.56	.787	1.185	113.67	1.957	22.08	303.92

PILE LOAD= 200.00 PENETRATION= 545.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.317	206.38	6.213	6.21	538.79
-.94	-.086	.197	211.79	3.194	9.41	535.59
1.06	.097	-.019	213.99	4.163	13.57	531.43
3.56	.327	.630	207.37	4.438	18.01	526.99
6.06	.557	1.106	185.74	3.938	21.94	523.06
8.56	.787	1.185	113.67	1.957	23.90	521.10

PILE LOAD= 218.00 PENETRATION= 903.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.705	219.18	6.563	6.56	896.44
-.94	-.086	-.041	222.60	3.369	9.93	893.07
1.06	.097	-.063	231.21	4.500	14.43	888.57
3.56	.327	1.407	232.97	4.938	19.37	883.63
6.06	.557	2.489	221.93	4.625	23.99	879.01
8.56	.787	.969	115.88	2.009	26.00	877.00

PILE LOAD= 225.00 PENETRATION= 1295.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.370	226.79	6.825	6.83	1288.18
1.06	.097	-.495	235.63	3.588	10.41	1284.59
-.94	-.086	-.063	231.21	4.500	14.91	1280.09
3.56	.327	.542	241.80	5.188	20.10	1274.90
6.06	.557	2.780	231.53	4.813	24.91	1270.09
8.56	.787	.752	118.09	2.060	26.97	1268.03

PILE LOAD= 235.00 PENETRATION= 1888.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.034	234.41	7.088	7.09	1880.91
-.94	-.086	-.279	233.42	3.544	10.63	1877.37
1.06	.097	-1.047	245.45	4.838	15.47	1872.53
3.56	.327	.207	249.42	5.375	20.84	1867.16
6.06	.557	2.758	240.14	5.000	25.84	1862.16
8.56	.787	.730	126.69	2.215	28.06	1859.94

PILE LOAD= 245.00 PENETRATION= 3140.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-.279	233.42	7.088	7.09	3132.91
-.94	-.086	-.204	245.23	3.719	10.81	3129.19
1.06	.097	-1.382	253.07	5.006	15.81	3124.19
3.56	.327	.185	258.03	5.563	21.38	3118.63
6.06	.557	3.363	250.73	5.188	26.56	3113.44
8.56	.787	1.238	134.09	2.318	28.88	3111.12

PILE LOAD= 160.00 PENETRATION= 3244.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-.708	153.96	4.725	4.73	3239.28
-.94	-.086	-.514	160.36	2.450	7.18	3236.83
1.06	.097	-1.693	168.20	3.375	10.55	3233.45
3.56	.327	.360	189.16	4.063	14.61	3229.39
6.06	.557	2.868	197.10	4.063	18.68	3225.33
8.56	.787	1.012	98.66	1.700	20.37	3223.63

PILE LOAD= 100.00 PENETRATION= 3220.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-.555	93.70	2.888	2.89	3217.11
-.94	-.086	-.458	96.90	1.488	4.38	3215.63

1.06	.097	-1.539	107.94	2.194	6.57	3213.43
3.56	.327	-.448	134.53	2.938	9.51	3210.49
6.06	.557	2.761	156.26	3.188	12.69	3207.31
8.56	.787	.646	77.25	1.339	14.03	3205.97

PILE LOAD= 0.00 PENETRATION= 2826.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-.530	1.22	.088	.09	2825.91
-.94	-.086	-1.545	-13.57	-.131	-.04	2826.04
1.06	.097	-1.589	3.65	.169	.13	2825.88
3.56	.327	-2.066	25.28	.688	.81	2825.19
6.06	.557	1.144	47.01	.938	1.75	2824.25
8.56	.787	3.401	112.00	1.803	3.55	2822.45

PILE LOAD= 0.00 PENETRATION= 2754.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-1.448	-10.37	-.175	-.18	2754.18
-.94	-.086	-.508	-7.39	-.088	-.26	2754.26
1.06	.097	-1.492	6.85	.225	-.04	2754.04
3.56	.327	-1.655	29.47	.750	.71	2753.29
6.06	.557	1.338	53.41	1.063	1.78	2752.23
8.56	.787	.411	4.19	.052	1.83	2752.17

PILE LOAD= 0.00 PENETRATION= 2747.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-1.642	-16.77	-.350	-.35	2747.35
-.94	-.086	-1.232	-12.58	-.131	-.48	2747.48
1.06	.097	-2.097	-3.74	.056	-.43	2747.43
3.56	.327	-1.536	24.07	.625	.20	2746.80
6.06	.557	1.144	47.01	.938	1.14	2745.86
8.56	.787	.843	-.22	-.052	1.09	2745.91

PILE LOAD= 0.00 PENETRATION= 2745.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	-1.448	-10.37	-.175	-.18	2745.18
-.94	-.086	-.508	-7.39	-.088	-.26	2745.26
1.06	.097	-1.567	-4.96	0.000	-.26	2745.26
3.56	.327	-1.006	22.85	.563	.30	2744.70
6.06	.557	.853	37.41	.750	1.05	2743.95
8.56	.787	1.329	15.78	.206	1.26	2743.74

PILE LOAD= 0.00 PENETRATION= 2748.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.097	3.20	.088	.09	2747.91
-.94	-.086	.313	.99	0.000	.09	2747.91
1.06	.097	-1.545	-13.57	-.169	-.08	2748.08
3.56	.327	-.596	27.04	.625	.54	2747.46
6.06	.557	.972	32.00	.625	1.17	2746.83
8.56	.787	.994	23.39	.361	1.53	2746.47

PILE LOAD= 0.00 PENETRATION= 2749.0

GDEP	DR	P	QH	DEF	SD	PED
-2.44	-.224	.097	3.20	.088	.09	2748.91
-.94	-.086	.411	4.19	.044	.13	2748.87
1.06	.097	-6.015	-160.77	-2.756	-2.63	2751.63
3.56	.327	-.476	21.63	.500	-2.13	2751.13
6.06	.557	-.097	-3.20	-.063	-2.19	2751.19
8.56	.787	-1.727	-66.21	-1.082	-3.27	2752.27

ERROR LC-2 IN STATEMENT 0001 + 00 LINES

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 06 MINUTES, 09 SECONDS.

..I 920130 MODEL PILE ROUTINE A M.C. HARRIS PLEASE SAVE OUTPUT CARDS
..LOAD FORGO CLOCK 600

TEST NO. 112 BASE DIA. 2.00 MAX LOAD 282.00
DEPTH= 11.23 SHAFT LENGTH= 10.20
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.380 POISSON RATIO= .3100

KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 13 NO. DEPTHS= 6

PILE LOAD= 40.00 PENETRATION= 12.5

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.107	40.83	1.225	1.23	11.28
-.59	-.053	.009	37.63	.569	1.79	10.71
1.41	.126	-.715	32.45	.675	2.47	10.03
3.91	.348	-.185	31.23	.688	3.16	9.34
6.41	.571	.150	23.62	.500	3.66	8.84
8.91	.793	.389	12.80	.206	3.86	8.64

PILE LOAD= 80.00 PENETRATION= 22.5

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.407	88.07	2.625	2.63	19.88
-.59	-.053	.116	78.47	1.181	3.81	18.69
1.41	.126	-.608	73.28	1.463	5.27	17.23
3.91	.348	.257	64.45	1.375	6.64	15.86
6.41	.571	.495	53.63	1.125	7.77	14.73
8.91	.793	.658	31.01	.515	8.28	14.22

PILE LOAD= 100.00 PENETRATION= 28.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.558	111.68	3.325	3.33	24.68
-.59	-.053	.266	102.08	1.531	4.86	23.14
1.41	.126	.169	98.88	1.913	6.77	21.23
3.91	.348	.505	91.27	1.938	8.71	19.29
6.41	.571	.160	61.25	1.313	10.02	17.98
8.91	.793	.539	36.42	.618	10.64	17.36

PILE LOAD= 140.00 PENETRATION= 40.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.373	142.92	4.288	4.29	35.71
-.59	-.053	.373	142.92	2.144	6.43	33.57
1.41	.126	-.859	130.34	2.588	9.02	30.98
3.91	.348	.028	112.90	2.438	11.46	28.54
6.41	.571	.527	82.66	1.750	13.21	26.79
8.91	.793	.398	50.43	.876	14.08	25.92

PILE LOAD= 180.00 PENETRATION= 66.0

GDEP	DR	P	QH	DEF	SD	PED

[illegible]

-2.09	-.186	.890	187.94	5.600	5.60	60.40
-.59	-.053	.285	177.35	2.669	8.27	57.73
1.41	.126	-.752	171.18	3.375	11.64	54.36
3.91	.348	.448	154.73	3.313	14.96	51.04
6.41	.571	1.185	113.67	2.375	17.33	48.67
8.91	.793	.743	80.45	1.391	18.72	47.28

PILE LOAD= 200.00 PENETRATION= 130.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	1.354	212.55	6.300	6.30	123.70
-.59	-.053	.630	207.37	3.106	9.41	120.59
1.41	.126	-1.229	192.81	3.825	13.23	116.77
3.91	.348	.599	178.34	3.813	17.04	112.96
6.41	.571	2.470	146.66	3.000	20.04	109.96
8.91	.793	1.304	108.26	1.854	21.90	108.10

PILE LOAD= 225.00 PENETRATION= 253.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	1.504	236.17	7.000	7.00	246.00
-.59	-.053	.878	234.19	3.500	10.50	242.50
1.41	.126	-1.197	221.83	4.388	14.89	238.11
3.91	.348	.414	209.58	4.500	19.39	233.61
6.41	.571	3.874	174.25	3.500	22.89	230.11
8.91	.793	1.379	120.07	2.060	24.95	228.05

PILE LOAD= 282.00 PENETRATION= 1355.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	1.059	286.83	8.575	8.58	1346.43
-.59	-.053	.357	273.04	4.113	12.69	1342.31
1.41	.126	-2.583	269.52	5.400	18.09	1336.91
3.91	.348	1.006	266.41	5.688	23.78	1331.23
6.41	.571	7.243	257.23	5.063	28.84	1326.16
8.91	.793	4.166	183.85	3.039	31.88	1323.12

PILE LOAD= 180.00 PENETRATION= 1806.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.793	184.74	5.513	5.51	1800.49
-.59	-.053	.793	184.74	2.756	8.27	1797.73
1.41	.126	-2.633	165.22	3.375	11.64	1794.36
3.91	.348	.069	179.56	3.875	15.52	1790.48
6.41	.571	6.457	193.99	3.750	19.27	1786.73
8.91	.793	2.439	117.64	1.957	21.23	1784.77

PILE LOAD= 100.00 PENETRATION= 1765.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.774	109.48	3.238	3.24	1761.76
-.59	-.053	.386	96.68	1.444	4.68	1760.32
1.41	.126	-1.830	98.34	2.025	6.71	1758.29
3.91	.348	-.501	114.12	2.500	9.21	1755.79
6.41	.571	5.216	143.78	2.750	11.96	1753.04
8.91	.793	2.526	83.20	1.339	13.30	1751.70

PILE LOAD= 0.00 PENETRATION= 1370.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	-.821	-8.38	-.175	-.18	1370.18
-.59	-.053	-.389	-12.80	-.175	-.35	1370.35
1.41	.126	-2.388	-13.34	-.113	-.46	1370.46
3.91	.348	-1.276	4.64	.188	-.28	1370.28
6.41	.571	2.150	24.16	.375	.10	1369.90
8.91	.793	1.081	-11.04	-.258	-.16	1370.16

PILE LOAD= 0.00 PENETRATION= 1270.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	-.097	-3.20	-.088	-.09	1270.09
-.59	-.053	.119	-5.41	-.088	-.18	1270.18
1.41	.126	-3.015	-15.33	-.113	-.29	1270.29
3.91	.348	-1.179	7.84	.250	-.04	1270.04
6.41	.571	2.344	30.56	.500	.46	1269.54
8.91	.793	1.373	-1.44	-.103	.36	1269.64

PILE LOAD= 0.00 PENETRATION= 1270.0

GDEP	DR	P	QH	DEF	SD	PED
-2.09	-.186	.216	-2.21	-.088	-.09	1270.09
-.59	-.053	.746	-3.42	-.088	-.18	1270.18
1.41	.126	-9.911	-37.15	-.113	-.29	1270.29
3.91	.348	-.887	17.44	.438	.15	1269.85
6.41	.571	1.642	16.77	.250	.40	1269.60
8.91	.793	.574	-18.43	-.361	.04	1269.96

ERROR LC-2 IN STATEMENT 0001 + 00 LINES

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 03 MINUTES, 41 SECONDS.

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

[illegible]

..I 920130 MODEL PILE ROUTINE A M.C. HARRIS PLEASE SAVE OUTPUT CARDS
..LOAD FORGO CLOCK 600
TEST NO. 121 BASE DIA. 1.0000
DEPTH= 11.92 SHAFT LENGTH= 11.12
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.380 POISSON RATIO= .3100

KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194
NO. PILE LOADS= 12 NO. DEPTHS= 6

PILE LOAD= 20.00 PENETRATION= 3.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	.755	34.21	.963	.96	2.04
.25	.021	.950	40.61	.569	1.53	1.47
2.25	.189	.150	23.62	.450	1.98	1.02
4.75	.398	-.119	5.41	.125	2.11	.89
7.25	.608	-.216	2.21	.063	2.17	.83
9.75	.818	.411	4.19	.052	2.22	.78

PILE LOAD= 40.00 PENETRATION= 5.2

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	.928	49.22	1.400	1.40	3.80
.25	.021	1.316	62.02	.875	2.28	2.93
2.25	.189	-.088	34.43	.675	2.95	2.25
4.75	.398	.053	20.42	.438	3.39	1.81
7.25	.608	-.238	10.82	.250	3.64	1.56
9.75	.818	.194	6.40	.103	3.74	1.46

PILE LOAD= 60.00 PENETRATION= 6.8

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	.981	69.63	2.013	2.01	4.79
.25	.021	1.078	72.84	1.050	3.06	3.74
2.25	.189	.063	58.05	1.125	4.19	2.61
4.75	.398	-.185	31.23	.688	4.88	1.93
7.25	.608	-.357	16.22	.375	5.25	1.55
9.75	.818	-.335	7.62	.155	5.40	1.40

PILE LOAD= 80.00 PENETRATION= 7.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.545	158.47	4.550	4.55	2.45
.25	.021	3.658	176.46	2.494	7.04	-.04
2.25	.189	.288	93.48	1.800	8.84	-1.84
4.75	.398	.204	44.03	.938	9.78	-2.78
7.25	.608	-.260	19.42	.438	10.22	-3.22
9.75	.818	-.746	3.42	.103	10.32	-3.32

PILE LOAD= 100.00 PENETRATION= 7.2

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.382	181.10	5.250	5.25	1.95
.25	.021	3.927	194.66	2.756	8.01	-.81

2.25	.189	.342	113.89	2.194	10.20	-3.00
4.75	.398	-.132	51.65	1.125	11.33	-4.13
7.25	.608	-.476	21.63	.500	11.83	-4.63
9.75	.818	-.649	6.62	.155	11.98	-4.78

PILE LOAD= 140.00 PENETRATION= 10.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.608	216.52	6.300	6.30	3.70
.25	.021	4.780	232.07	3.281	9.58	.42
2.25	.189	-.232	132.33	2.588	12.17	-2.17
4.75	.398	-.078	72.07	1.563	13.73	-3.73
7.25	.608	-.185	31.23	.688	14.42	-4.42
9.75	.818	-.649	6.62	.155	14.57	-4.57

PILE LOAD= 180.00 PENETRATION= 17.8

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.087	255.37	7.525	7.53	10.28
.25	.021	5.081	279.31	3.981	11.51	6.29
2.25	.189	-.320	166.76	3.263	14.77	3.03
4.75	.398	-.458	96.90	2.125	16.89	.91
7.25	.608	-.110	43.04	.938	17.83	-.03
9.75	.818	-.962	5.63	.155	17.99	-.19

PILE LOAD= 200.00 PENETRATION= 29.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.865	280.97	8.225	8.23	20.78
.25	.021	5.761	301.71	4.288	12.51	16.49
2.25	.189	.144	191.37	3.713	16.23	12.78
4.75	.398	-.599	110.92	2.438	18.66	10.34
7.25	.608	-.013	46.24	1.000	19.66	9.34
9.75	.818	-1.805	5.86	.206	19.87	9.13

PILE LOAD= 100.00 PENETRATION= 232.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	3.711	196.87	5.600	5.60	226.40
.25	.021	6.726	212.20	2.888	8.49	223.51
2.25	.189	2.125	116.64	2.138	10.63	221.38
4.75	.398	-.251	57.06	1.250	11.88	220.13
7.25	.608	1.016	14.78	.250	12.13	219.88
9.75	.818	-2.291	-10.14	-.052	12.07	219.93

PILE LOAD= 0.00 PENETRATION= 218.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	3.153	85.19	2.275	2.28	215.73
.25	.021	4.115	79.56	1.006	3.28	214.72
2.25	.189	2.733	43.36	.675	3.96	214.04
4.75	.398	-.940	-2.98	0.000	3.96	214.04
7.25	.608	.066	-25.83	-.563	3.39	214.61
9.75	.818	-2.777	-26.15	-.309	3.08	214.92

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PILE LOAD= 0.00 PENETRATION= 216.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	2.721	89.60	2.450	2.45	213.55
.25	.021	5.400	112.55	1.444	3.89	212.11
2.25	.189	2.323	39.17	.619	4.51	211.49
4.75	.398	-.843	.22	.063	4.58	211.43
7.25	.608	-.053	-20.42	-.438	4.14	211.86
9.75	.818	-3.307	-24.93	-.258	3.88	212.12

PILE LOAD= 0.00 PENETRATION= 217.0

GDEP	DR	P	QH	DEF	SD	PED
-1.25	-.105	4.460	109.57	2.888	2.89	214.11
.25	.021	5.357	129.77	1.706	4.59	212.41
2.25	.189	1.987	46.79	.788	5.38	211.62
4.75	.398	-.768	12.03	.313	5.69	211.31
7.25	.608	-.075	-11.81	-.250	5.44	211.56
9.75	.818	-3.545	-14.11	-.052	5.39	211.61

TEST NO. 122 BASE DIA. 1.0000
DEPTH= 11.97 SHAFT LENGTH= 11.17
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.380 POISSON RATIO= .3100

KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194
NO. PILE LOADS= 11 NO. DEPTHS= 6

PILE LOAD= 40.00 PENETRATION= 6.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.107	40.83	1.225	1.23	4.78
.55	.046	.733	42.82	.613	1.84	4.16
2.55	.213	-.282	28.03	.563	2.40	3.60
5.05	.422	.053	20.42	.438	2.84	3.16
7.55	.631	-.357	16.22	.375	3.21	2.79
10.05	.840	.291	9.60	.155	3.37	2.63

PILE LOAD= 80.00 PENETRATION= 8.8

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.213	81.67	2.450	2.45	6.35
.55	.046	.646	77.25	1.138	3.59	5.21
2.55	.213	-.154	60.26	1.181	4.77	4.03
5.05	.422	-.737	41.06	.938	5.71	3.09
7.55	.631	.031	29.03	.625	6.33	2.47
10.05	.840	-.454	13.02	.258	6.59	2.21

PILE LOAD= 100.00 PENETRATION= 10.8

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.169	98.88	2.975	2.98	7.83
.55	.046	1.229	96.45	1.400	4.38	6.43
2.55	.213	-.295	74.28	1.463	5.84	4.96
5.05	.422	.279	55.84	1.188	7.03	3.78
7.55	.631	.323	38.63	.813	7.84	2.96
10.05	.840	-.141	14.02	.258	8.10	2.71

PILE LOAD= 140.00 PENETRATION= 15.2

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.276	139.72	4.200	4.20	11.00
.55	.046	1.746	141.48	2.056	6.26	8.94
2.55	.213	-.382	108.71	2.138	8.39	6.81
5.05	.422	-.078	72.07	1.563	9.96	5.24
7.55	.631	-.110	43.04	.938	10.89	4.31
10.05	.840	-.454	13.02	.258	11.15	4.05

PILE LOAD= 161.00 PENETRATION= 93.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.912	179.34	5.338	5.34	87.66
.55	.046	3.539	181.86	2.581	7.92	85.08
2.55	.213	.903	141.70	2.700	10.62	82.38

5.05	.422	.364	105.28	2.250	12.87	80.13
7.55	.631	2.019	75.81	1.500	14.37	78.63
10.05	.840	-.617	35.65	.670	15.04	77.96

PILE LOAD= 220.00 PENETRATION= 853.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	1.504	236.17	7.000	7.00	846.00
.55	.046	5.601	240.46	3.369	10.37	842.63
2.55	.213	1.906	202.73	3.825	14.19	838.81
5.05	.422	1.345	174.92	3.688	17.88	835.12
7.55	.631	4.426	164.42	3.250	21.13	831.87
10.05	.840	1.414	65.22	1.082	22.21	830.79

PILE LOAD= 120.00 PENETRATION= 1103.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	-.016	130.12	3.938	3.94	1099.06
.55	.046	5.626	147.97	1.969	5.91	1097.09
2.55	.213	1.693	121.06	2.250	8.16	1094.84
5.05	.422	.461	108.48	2.313	10.47	1092.53
7.55	.631	4.871	113.76	2.125	12.59	1090.41
10.05	.840	.777	25.60	.412	13.01	1089.99

PILE LOAD= 40.00 PENETRATION= 1093.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.614	48.23	1.400	1.40	1091.60
.55	.046	4.786	64.32	.744	2.14	1090.86
2.55	.213	2.062	58.59	1.013	3.16	1089.84
5.05	.422	-.034	54.85	1.188	4.34	1088.66
7.55	.631	4.548	75.14	1.313	5.66	1087.34
10.05	.840	.313	.99	0.000	5.66	1087.34

PILE LOAD= 0.00 PENETRATION= 1086.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.097	3.20	.088	.09	1085.91
.55	.046	4.366	22.50	.131	.22	1085.78
2.55	.213	.896	20.19	.338	.56	1085.44
5.05	.422	-.066	25.83	.563	1.12	1084.88
7.55	.631	4.495	54.72	.875	1.99	1084.01
10.05	.840	-.075	-11.81	-.206	1.79	1084.21

PILE LOAD= 0.00 PENETRATION= 1076.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.508	7.39	.175	.18	1075.83
.55	.046	2.388	13.34	.088	.26	1075.74
2.55	.213	.777	25.60	.450	.71	1075.29
5.05	.422	-.790	20.64	.500	1.21	1074.79
7.55	.631	2.733	43.36	.750	1.96	1074.04
10.05	.840	-.746	3.42	.103	2.07	1073.93

PILE LOAD= 0.00 PENETRATION= 1075.0

GDEP	DR	P	QH	DEF	SD	PED
-.95	-.079	.194	6.40	.175	.18	1074.83
.55	.046	2.075	12.35	.088	.26	1074.74
2.55	.213	.755	34.21	.619	.88	1074.12
5.05	.422	-.693	23.84	.563	1.44	1073.56
7.55	.631	2.733	43.36	.750	2.19	1072.81
10.05	.840	-.530	1.22	.052	2.25	1072.75

ERROR LC-2 IN STATEMENT 0001 + 00 LINES

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 06 MINUTES, 33 SECONDS.

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..I 920130 MODEL PILE  ROUTINE A  M.C. HARRIS  PLEASE SAVE OUTPUT CARDS
..LOAD FORGO CLOCK 600
 TEST NO. 130  BASE DIA.    2.0000
 DEPTH=  11.90     SHAFT LENGTH=  10.75
 PILE AREA=    1.0700    THICK RATIO=      .1048
 MOD ELAST PILE= 15.380  POISSON RATIO=  .3100

 KA=       .5982  KB=       .1854  KC=       .6400
 KD=       .1984  KE=       .0627  KF=       .0194
 NO. PILE LOADS=  16      NO. DEPTHS=   6
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PILE LOAD= 20.00   PENETRATION= 2.5

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	-.044	17.22	.525	.53	1.98
.08	.007	-.357	16.22	.263	.79	1.71
2.08	.175	.150	23.62	.450	1.24	1.26
4.58	.385	-.238	10.82	.250	1.49	1.01
7.08	.595	.075	11.81	.250	1.74	.76
9.58	.805	-.433	4.42	.103	1.84	.66

PILE LOAD= 40.00   PENETRATION= 4.2

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.107	40.83	1.225	1.23	2.98
.08	.007	-.088	34.43	.525	1.75	2.45
2.08	.175	.204	44.03	.844	2.59	1.61
4.58	.385	-.379	24.83	.563	3.16	1.04
7.08	.595	.270	18.21	.375	3.53	.67
9.58	.805	-.238	10.82	.206	3.74	.46

PILE LOAD= 60.00   PENETRATION= 6.3

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.160	61.25	1.838	1.84	4.46
.08	.007	.063	58.05	.875	2.71	3.59
2.08	.175	.473	62.24	1.181	3.89	2.41
4.58	.385	.107	40.83	.875	4.77	1.53
7.08	.595	.464	24.61	.500	5.27	1.03
9.58	.805	-.552	9.82	.206	5.47	.83

PILE LOAD= 80.00   PENETRATION= 10.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.646	77.25	2.275	2.28	7.73
.08	.007	.332	76.26	1.138	3.41	6.59
2.08	.175	.116	78.47	1.519	4.93	5.07
4.58	.385	-.034	54.85	1.188	6.12	3.88
7.08	.595	.539	36.42	.750	6.87	3.13
9.58	.805	.150	23.62	.412	7.28	2.72

PILE LOAD= 100.00   PENETRATION= 15.9

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.169	98.88	2.975	2.98	12.93
.08	.007	.169	98.88	1.488	4.46	11.44

2.08	.175	-.122	89.28	1.744	6.21	9.69
4.58	.385	-.273	65.67	1.438	7.64	8.26
7.08	.595	.107	40.83	.875	8.52	7.38
9.58	.805	-.476	21.63	.412	8.93	6.97

PILE LOAD= 120.00 PENETRATION= 21.7

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.849	121.29	3.588	3.59	18.11
.08	.007	.752	118.09	1.750	5.34	16.36
2.08	.175	.536	120.29	2.306	7.64	14.06
4.58	.385	.743	80.45	1.688	9.33	12.37
7.08	.595	.398	50.43	1.063	10.39	11.31
9.58	.805	.031	29.03	.515	10.91	10.79

PILE LOAD= 140.00 PENETRATION= 27.5

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.081	133.32	4.025	4.03	23.48
.08	.007	.708	135.30	2.013	6.04	21.46
2.08	.175	-.016	130.12	2.531	8.57	18.93
4.58	.385	.169	98.88	2.125	10.69	16.81
7.08	.595	.376	59.04	1.250	11.94	15.56
9.58	.805	-.088	34.43	.618	12.56	14.94

PILE LOAD= 160.00 PENETRATION= 36.8

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	.859	158.92	4.725	4.73	32.08
.08	.007	.978	153.51	2.275	7.00	29.80
2.08	.175	.254	148.33	2.869	9.87	26.93
4.58	.385	.028	112.90	2.438	12.31	24.49
7.08	.595	.743	80.45	1.688	13.99	22.81
9.58	.805	-.207	39.84	.721	14.71	22.09

PILE LOAD= 180.00 PENETRATION= 47.1

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	1.323	183.53	5.425	5.43	41.68
.08	.007	1.636	184.52	2.713	8.14	38.96
2.08	.175	-.223	169.96	3.319	11.46	35.64
4.58	.385	.179	136.52	2.938	14.39	32.71
7.08	.595	.699	97.67	2.063	16.46	30.64
9.58	.805	-.445	50.66	.927	17.38	29.72

PILE LOAD= 200.00 PENETRATION= 65.4

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	1.689	204.94	6.038	6.04	59.36
.08	.007	1.787	208.14	3.063	9.10	56.30
2.08	.175	-.072	193.58	3.769	12.87	52.53
4.58	.385	.643	161.13	3.438	16.31	49.09
7.08	.595	1.260	125.48	2.625	18.93	46.47
9.58	.805	.019	75.27	1.339	20.27	45.13

PILE LOAD= 543.00 PENETRATION= 5875.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	2.326	533.81	15.925	15.93	5859.08
.08	.007	5.871	547.93	8.006	23.93	5851.07
2.08	.175	.639	534.26	10.350	34.28	5840.72
4.58	.385	.564	522.45	11.250	45.53	5829.47
7.08	.595	-3.379	457.91	10.125	55.66	5819.34
9.58	.805	2.990	397.07	6.901	62.56	5812.44

PILE LOAD= 300.00 PENETRATION= 6105.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	2.075	301.61	8.925	8.93	6096.08
.08	.007	2.918	301.39	4.419	13.34	6091.66
2.08	.175	-10.528	287.76	6.244	19.59	6085.41
4.58	.385	-4.799	271.18	6.188	25.78	6079.23
7.08	.595	-4.379	313.01	7.063	32.84	6072.16
9.58	.805	-3.836	265.55	4.944	37.78	6067.22

PILE LOAD= 0.00 PENETRATION= 5670.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	-1.642	-16.77	-.350	-.35	5670.35
.08	.007	1.903	-2.66	-.131	-.48	5670.48
2.08	.175	-2.649	6.08	.281	-.20	5670.20
4.58	.385	-3.200	15.90	.563	.36	5669.64
7.08	.595	-3.394	9.50	.438	.80	5669.20
9.58	.805	-4.322	-39.71	-.464	.34	5669.66

PILE LOAD= 0.00 PENETRATION= 5619.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	-.702	-13.79	-.350	-.35	5619.35
.08	.007	2.291	10.14	.044	-.31	5619.31
2.08	.175	-.909	26.05	.563	.26	5618.74
4.58	.385	-1.677	38.08	.938	1.19	5617.81
7.08	.595	-2.573	17.89	.563	1.76	5617.24
9.58	.805	-1.783	-2.75	.052	1.81	5617.19

PILE LOAD= 0.00 PENETRATION= 5613.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	-1.210	-21.18	-.525	-.53	5613.53
.08	.007	2.291	10.14	.044	-.48	5613.48
2.08	.175	-1.103	19.65	.450	-.03	5613.03
4.58	.385	-1.677	38.08	.938	.91	5612.09
7.08	.595	-3.156	-1.31	.188	1.09	5611.91
9.58	.805	-3.253	-4.51	.103	1.20	5611.80

PILE LOAD= 0.00 PENETRATION= 5613.0

GDEP	DR	P	QH	DEF	SD	PED
-1.42	-.119	-.702	-13.79	-.350	-.35	5613.35
.08	.007	2.075	12.35	.088	-.26	5613.26
2.08	.175	-1.395	10.05	.281	.02	5612.98
4.58	.385	-2.087	33.89	.875	.89	5612.11

[illegible]

7.08	.595	-3.156	-1.31	.188	1.08	5611.92
9.58	.805	-3.156	-1.31	.155	1.24	5611.76

TEST NO. 140 BASE DIA. 1.0000
DEPTH= 12.35 SHAFT LENGTH= 11.55
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.380 POISSON RATIO= .3100

KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194
NO. PILE LOADS= 10 NO. DEPTHS= 6

PILE LOAD= 20.00 PENETRATION= 6.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	.367	21.41	.613	.61	5.39
.91	.074	.150	23.62	.350	.96	5.04
2.91	.236	.367	21.41	.394	1.36	4.64
5.41	.438	.389	12.80	.250	1.61	4.39
7.91	.640	.291	9.60	.188	1.79	4.21
10.41	.843	-.530	1.22	.052	1.85	4.15

PILE LOAD= 40.00 PENETRATION= 11.4

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	.636	39.62	1.138	1.14	10.26
.91	.074	.575	46.95	.683	1.82	9.58
2.91	.236	.614	48.23	.900	2.72	8.68
5.41	.438	.972	32.00	.625	3.35	8.06
7.91	.640	.486	16.00	.313	3.66	7.74
10.41	.843	1.016	14.78	.206	3.86	7.54

PILE LOAD= 60.00 PENETRATION= 17.5

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	1.003	61.03	1.750	1.75	15.75
.91	.074	.787	63.23	.919	2.67	14.83
2.91	.236	.138	69.86	1.350	4.02	13.48
5.41	.438	.398	50.43	1.063	5.08	12.42
7.91	.640	.561	27.81	.563	5.64	11.86
10.41	.843	.172	15.01	.258	5.90	11.60

PILE LOAD= 80.00 PENETRATION= 25.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	.862	75.04	2.188	2.19	22.81
.91	.074	1.251	87.84	1.269	3.46	21.54
2.91	.236	.937	86.85	1.631	5.09	19.91
5.41	.438	.981	69.63	1.438	6.53	18.48
7.91	.640	.517	45.03	.938	7.46	17.54
10.41	.843	.874	28.80	.464	7.93	17.07

PILE LOAD= 100.00 PENETRATION= 37.4

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	.796	100.87	2.975	2.98	34.43
.91	.074	1.834	107.04	1.531	4.51	32.89
2.91	.236	.050	104.29	2.025	6.53	30.87

5.41	.438	.721	89.06	1.875	8.41	28.99
7.91	.640	.690	60.03	1.250	9.66	27.74
10.41	.843	.539	36.42	.618	10.27	27.13

PILE LOAD= 114.80 PENETRATION= 1043.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	2.081	133.86	3.850	3.85	1039.15
.91	.074	6.156	146.76	1.925	5.78	1037.23
2.91	.236	.925	133.09	2.531	8.31	1034.69
5.41	.438	-.113	126.92	2.750	11.06	1031.94
7.91	.640	.677	106.28	2.250	13.31	1029.69
10.41	.843	.624	85.86	1.494	14.80	1028.20

PILE LOAD= 60.00 PENETRATION= 1507.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	3.586	80.77	2.100	2.10	1504.90
.91	.074	5.661	93.12	1.138	3.24	1503.76
2.91	.236	-.348	53.86	1.069	4.31	1502.69
5.41	.438	.094	87.08	1.875	6.18	1500.82
7.91	.640	.743	80.45	1.688	7.87	1499.13
10.41	.843	-.273	65.67	1.185	9.05	1497.95

PILE LOAD= 0.00 PENETRATION= 1480.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	-.552	9.82	.350	.35	1479.65
.91	.074	5.284	34.08	.263	.61	1479.39
2.91	.236	.539	36.42	.675	1.29	1478.71
5.41	.438	.495	53.63	1.125	2.41	1477.59
7.91	.640	.711	51.43	1.063	3.48	1476.53
10.41	.843	-.034	54.85	.979	4.45	1475.55

PILE LOAD= 0.00 PENETRATION= 1473.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	-.433	4.42	.175	.18	1472.83
.91	.074	3.934	26.91	.219	.39	1472.61
2.91	.236	-.433	4.42	.113	.51	1472.49
5.41	.438	.009	37.63	.813	1.32	1471.68
7.91	.640	.226	35.43	.750	2.07	1470.93
10.41	.843	-.034	54.85	.979	3.05	1469.95

PILE LOAD= 0.00 PENETRATION= 1472.0

GDEP	DR	P	QH	DEF	SD	PED
-.59	-.048	.821	8.38	.175	.18	1471.83
.91	.074	3.836	23.71	.175	.35	1471.65
2.91	.236	-1.016	-14.78	-.225	.13	1471.88
5.41	.438	-.185	31.23	.688	.81	1471.19
7.91	.640	.129	32.23	.688	1.50	1470.50
10.41	.843	.107	40.83	.721	2.22	1469.78

ERROR LC-2 IN STATEMENT 0001 + 00 LINES

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 07 MINUTES, 22 SECONDS.

[illegible]

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Routine B

TEST NO. 101 BASE DIA. 2.00
DEPTH= 16.120INCHES
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.3800 POISSON RATIO= .3100
KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 7 NO. DEPTHS= 6

PILE LOAD= 50.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.182	52.64	1.053	45.00
2.000	.376	59.04	1.181	50.00
3.000	.646	77.25	1.545	65.00
4.000	.570	65.44	1.309	55.00
5.000	.398	50.43	1.009	42.50
6.000	-.238	10.82	.216	10.00

PILE LOAD= 100.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.263	103.30	1.033	90.00
2.000	.050	104.29	1.043	90.00
3.000	.201	127.91	1.279	110.00
4.000	.244	110.69	1.107	95.00
5.000	-.436	88.29	.883	77.50
6.000	-3.877	-90.37	-.904	-67.50

PILE LOAD= 150.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.254	140.93	.940	122.50
2.000	-.686	145.35	.969	127.50
3.000	-.461	180.78	1.205	157.50
4.000	-.903	147.56	.984	130.00
5.000	-.762	133.54	.890	117.50
6.000	-5.259	-126.57	-.844	-95.00

PILE LOAD= 200.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-1.423	186.41	.932	165.00
2.000	-1.034	199.21	.996	175.00
3.000	-1.316	227.24	1.136	200.00
4.000	-.840	205.61	1.028	180.00
5.000	-2.028	175.82	.879	157.50
6.000	-6.686	-145.54	-.728	-107.50

PILE LOAD= 300.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-1.157	288.49	.962	252.50
2.000	-1.686	289.71	.966	255.00
3.000	-2.207	328.56	1.095	290.00

4.000	-1.708	298.32	.994	262.50
5.000	-2.680	266.32	.888	237.50
6.000	-7.714	-114.08	-.380	-77.50

PILE LOAD= 400.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-2.630	370.61	.927	327.50
2.000	-2.652	379.22	.948	335.00
3.000	-3.507	425.68	1.064	377.50
4.000	-1.874	404.82	1.012	355.00
5.000	-3.764	361.23	.903	322.50
6.000	-8.278	-58.02	-.145	-27.50

PILE LOAD= 500.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-2.871	465.30	.931	410.00
2.000	-3.520	471.92	.944	417.50
3.000	-4.711	526.01	1.052	467.50
4.000	-3.347	486.93	.974	430.00
5.000	-4.416	451.73	.903	402.50
6.000	-8.961	3.46	.007	27.50

TEST NO. 102 BASE DIA. 2.00
DEPTH= 16.120INCHES
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.3800 POISSON RATIO= .3100
KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 7 NO. DEPTHS= 6

PILE LOAD= 50.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	1.100	64.23	1.285	52.50
2.000	1.003	61.03	1.221	50.00
3.000	.981	69.63	1.393	57.50
4.000	.787	63.23	1.265	52.50
5.000	.592	56.83	1.137	47.50
6.000	1.197	67.43	1.349	55.00

PILE LOAD= 100.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.677	106.28	1.063	90.00
2.000	.774	109.48	1.095	92.50
3.000	.244	110.69	1.107	95.00
4.000	.461	108.48	1.085	92.50
5.000	.169	98.88	.989	85.00
6.000	2.526	83.20	.832	65.00

PILE LOAD= 150.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.157	144.13	.961	125.00
2.000	-.060	147.33	.982	127.50
3.000	-.925	156.17	1.041	137.50
4.000	-.395	154.95	1.033	135.00
5.000	-1.194	137.96	.920	122.50
6.000	2.385	97.22	.648	77.50

PILE LOAD= 200.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.386	192.58	.963	167.50
2.000	-.602	194.79	.974	170.00
3.000	-1.251	201.42	1.007	177.50
4.000	-.624	203.40	1.017	177.50
5.000	-1.931	179.02	.895	160.00
6.000	1.887	127.46	.637	105.00

PILE LOAD= 300.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-1.664	281.10	.937	247.50
2.000	-1.254	285.29	.951	250.00
3.000	-2.238	299.53	.998	265.00

4.000	-1.179	297.10	.990	260.00
5.000	-2.799	271.72	.906	242.50
6.000	.846	205.16	.684	175.00

PILE LOAD= 400.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-1.787	370.38	.926	325.00
2.000	-3.786	369.84	.925	330.00
3.000	-4.046	389.26	.973	347.50
4.000	-1.950	393.01	.983	345.00
5.000	-4.391	359.25	.898	322.50
6.000	.530	288.04	.720	247.50

PILE LOAD= 500.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-3.065	458.90	.918	405.00
2.000	-4.222	458.13	.916	407.50
3.000	-5.639	476.79	.954	427.50
4.000	-2.191	487.70	.975	427.50
5.000	-5.043	449.75	.899	402.50
6.000	.191	379.54	.759	327.50

TEST NO. 103 BASE DIA. 1.00
DEPTH= 16.120INCHES
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.3800 POISSON RATIO= .3100
KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 9 NO. DEPTHS= 6

PILE LOAD= 20.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.282	28.03	1.402	25.00
2.000	.561	27.81	1.390	22.50
3.000	.561	27.81	1.390	22.50
4.000	.561	27.81	1.390	22.50
5.000	.777	25.60	1.280	20.00
6.000	.583	19.20	.960	15.00

PILE LOAD= 50.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	1.608	71.62	1.432	57.50
2.000	1.003	61.03	1.221	50.00
3.000	.473	62.24	1.245	52.50
4.000	.570	65.44	1.309	55.00
5.000	.495	53.63	1.073	45.00
6.000	.896	20.19	.404	15.00

PILE LOAD= 75.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	1.034	90.05	1.201	75.00
2.000	.721	89.06	1.187	75.00
3.000	.527	82.66	1.102	70.00
4.000	.624	85.86	1.145	72.50
5.000	.332	76.26	1.017	65.00
6.000	1.599	33.99	.453	25.00

PILE LOAD= 100.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	1.185	113.67	1.137	95.00
2.000	.461	108.48	1.085	92.50
3.000	.558	111.68	1.117	95.00
4.000	.752	118.09	1.181	100.00
5.000	.580	103.08	1.031	87.50
6.000	1.674	45.79	.458	35.00

PILE LOAD= 150.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.956	162.12	1.081	137.50
2.000	.232	156.93	1.046	135.00
3.000	-.298	158.15	1.054	137.50

4.000	.307	168.74	1.125	145.00
5.000	-.373	146.34	.976	127.50
6.000	-21.295	7.81	.052	65.00

PILE LOAD= 200.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.630	207.37	1.037	177.50
2.000	-.505	197.99	.990	172.50
3.000	-1.153	204.62	1.023	180.00
4.000	-.332	213.00	1.065	185.00
5.000	-.602	194.79	.974	170.00
6.000	.969	115.88	.579	97.50

PILE LOAD= 300.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.097	292.46	.975	252.50
2.000	-1.179	297.10	.990	260.00
3.000	-2.335	296.33	.988	262.50
4.000	-.379	314.09	1.047	272.50
5.000	-1.351	282.09	.940	247.50
6.000	.436	200.97	.670	172.50

PILE LOAD= 400.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.241	383.95	.960	332.50
2.000	-2.144	386.61	.967	340.00
3.000	-3.830	387.06	.968	345.00
4.000	-1.247	406.80	1.017	355.00
5.000	-2.219	374.80	.937	330.00
6.000	.508	296.65	.742	255.00

PILE LOAD= 500.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.047	477.43	.955	412.50
2.000	-3.109	476.12	.952	420.00
3.000	-5.444	483.19	.966	432.50
4.000	-2.429	498.52	.997	437.50
5.000	-3.617	468.72	.937	415.00
6.000	.266	391.34	.783	337.50

TEST NO. 104 BASE DIA. 2.00
DEPTH= 16.120INCHES
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.3800 POISSON RATIO= .3100
KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 4 NO. DEPTHS= 6

PILE LOAD= 20.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.658	31.01	1.550	25.00
2.000	.248	26.82	1.341	22.50
3.000	.248	26.82	1.341	22.50
4.000	-.066	25.83	1.291	22.50
5.000	.053	20.42	1.021	17.50
6.000	.150	23.62	1.181	20.00

PILE LOAD= 50.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	.354	67.65	1.353	57.50
2.000	.063	58.05	1.161	50.00
3.000	.160	61.25	1.225	52.50
4.000	.473	62.24	1.245	52.50
5.000	-.132	51.65	1.033	45.00
6.000	1.025	52.42	1.048	42.50

PILE LOAD= 100.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.069	109.70	1.097	95.00
2.000	-.360	100.10	1.001	87.50
3.000	-.674	99.11	.991	87.50
4.000	.147	107.49	1.075	92.50
5.000	-.144	97.89	.979	85.00
6.000	2.213	82.21	.822	65.00

PILE LOAD= 200.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-.213	207.59	1.038	180.00
2.000	-.915	193.80	.969	170.00
3.000	-1.661	197.22	.986	175.00
4.000	-.332	213.00	1.065	185.00
5.000	-1.423	186.41	.932	165.00
6.000	2.567	149.86	.749	122.50

TEST NO. 201 BASE DIA. 1.00
DEPTH= 15.500INCHES
PILE AREA= 1.0700 THICK RATIO= .1048
MOD ELAST PILE= 15.3800 POISSON RATIO= .3100
KA= .5982 KB= .1854 KC= .6400
KD= .1984 KE= .0627 KF= .0194

NO. PILE LOADS= 6 NO. DEPTHS= 6

PILE LOAD= 100.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-3.548	69.76	.698	70.00
2.000	-3.116	65.35	.653	65.00
3.000	-3.548	69.76	.698	70.00
4.000	-6.056	61.83	.618	70.00
5.000	-4.952	42.18	.422	50.00
6.000	.759	-49.67	-.497	-45.00

PILE LOAD= 200.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-4.394	153.86	.769	145.00
2.000	-5.216	145.48	.727	140.00
3.000	-4.200	160.26	.801	150.00
4.000	-9.215	144.39	.722	150.00
5.000	-5.993	119.88	.599	120.00
6.000	1.016	14.78	.074	10.00

PILE LOAD= 300.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-5.867	235.98	.787	220.00
2.000	-6.689	227.59	.759	215.00
3.000	-6.927	238.41	.795	225.00
4.000	-9.434	230.47	.768	225.00
5.000	-8.287	193.61	.645	190.00
6.000	-.176	68.87	.230	60.00

PILE LOAD= 400.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-6.908	313.68	.784	290.00
2.000	-7.341	318.09	.795	295.00
3.000	-8.833	324.94	.812	305.00
4.000	-11.967	315.02	.788	305.00
5.000	-11.403	258.96	.647	255.00
6.000	-1.022	152.97	.382	135.00

PILE LOAD= 500.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-6.933	406.16	.812	370.00
2.000	-8.187	402.19	.804	370.00
3.000	-8.858	417.43	.835	385.00

4.000	-12.619	405.52	.811	385.00
5.000	-13.352	362.70	.725	350.00
6.000	-.614	241.03	.482	210.00

PILE LOAD= 600.00KGMS.

GDEP	P	QH	QR	(SVL+SVR)/2
1.000	-8.406	488.28	.814	445.00
2.000	-9.271	497.11	.829	455.00
3.000	-11.453	443.92	.740	415.00
4.000	-12.450	504.41	.841	470.00
5.000	-12.124	459.16	.765	430.00
6.000	5.867	342.54	.571	280.00

ERROR LC-2 IN STATEMENT 0076 + 00 LINES

RUNNING TIME FOR THIS PROGRAM WAS 0000 HOURS, 08 MINUTES, 18 SECONDS.